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The Bond Between Concrete and Steel*

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SYNOPSIS

To provide the concrete designer with the background for judging whether or not bond specifications have kept pace with the changes in reinforced concrete materials and practice, this paper presents the major problems of bond and discusses them in the light of the varied, uncoordinated and sometimes conflicting, evidence available. While the more recent investigations, including extensive researches by the authors, show that some of the most widely accepted concepts, notably those relating to the proportionality of bond resistance to strength of concrete and to length of embedment, are fallaceous, tests are reassuring in their indication that factors of safety against bond failures compare favorably with those for diagonal tension in the concrete and tension in the steel. Another useful indication is that the vertically cast pullout type of specimen is a good criterion for the bond resistance in a horizontally cast beam in spite of differences in actual bond resistance because of such factors as honeycombing and the water-gain always more or less present below a horizontal bar. Discussions are necessarily brief and wherever possible, references are given to specific or more complete data. Obviously, there remain important unanswered questions that must await the results from further research.

INTRODUCTION

When one attempts to itemize what is known about bond and anchorage, he finds, among the scattered researches into the field, much that is incomplete or contradictory and much more that is inapplicable to present day concretes. The classic work of Abrams[3] constitututed a remarkable one-man symposium upon bond as applied to the bars and mixtures of that day. While much of Bulletin

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(3)See list of references, p. 19.

71 [3] is as valid now as it was twenty-five years ago, there are important concepts that need revision in the light of tests on current mixtures.

Among the items which may influence potential bond resistance are: strength of concrete, length of embedment, use of hooks or bends, nature of the stress in the concrete surrounding the bar, deformations along the bar, lugs, surface texture of bar, twisting of bars, vibration of concrete during or following placement, orientation of bar as cast (vertical or horizontal), condition of concrete as loaded (dry or saturated), method of test, intensity of the steel stress, and possibly the type of cement, admixture, or aggregate used. Environmental conditions for the hardened concrete such as alternate freezing and thawing, alternately wet and dry, and subjection to a wide temperature variation may well be important because all of these conditions swell and shrink the concrete, producing, thereby, differential motions tending to break the bond.

CURRENT QUESTIONS

Since the early bond tests of Withey [1] [2] and of Abrams [3], the strength of structural concrete has doubled and working stresses in the steel have been substantially increased. Old unanswered, or partially answered, questions on bond have taken on added importance and new questions have arisen. With increasing frequency one hears raised such queries as the following:

1. Does doubling the compressive strength of concrete double its potential bond resistance?
2. Does doubling the length of embedment of a plain reinforcing bar double its resistance to slippage?
3. How much and under what conditions does the bending of bars influence the effectiveness of their anchorage?
4. Is the effectiveness of bond markedly different for embedments which are in a region of compressive concrete than it is when the embedment is in a region of tension?
5. How about the relative effectiveness of anchorage adjacent to or remote from the region of its immediate need, for developing tensile resistance in the steel?
6. How do deformed bars compare with plain bars in resistance to slippage?
7. What are the effects of surface coatings, and especially of rust in its varying degrees of depth and firmness?
8. How is bond resistance influenced by different surface textures such as those secured by rolling vs. drawing?
9. How much is provided in added effectiveness of bond resistance by the welded intersections of a mesh or mat?
10. Does twisting two small plain bars together produce a type of bond differing in nature or magnitude from that of the square twisted bars tested by Abrams years ago?[3]*

*(Reference 3, p. 73)

11. When testing specimens, whether pullout or pushout specimens or beams, in the usual types of testing machines, the load falls off as slip occurs because it cannot follow down as it would in the case of a load resting on the beam or hung on the bar of a pullout specimen. Does this action invalidate the results of such laboratory bond tests as criteria of the bond resistance of bars in structural members?

12. Steel is used for both tensile and compressive reinforcement in concrete. Is bond essentially the same problem in the two types of stressing?

13. Does the intensity of tensile stress in the steel, because of its tendency to draw down the bar or decrease it slightly in cross section, or because of the localized progressive slippage from the longitudinal strain, influence the bond resistance that can be developed by a plain bar: (a) at the yield point? (b) below the yield point?

14. There is water gain on the underside of all surfaces that are horizontal as cast. Does this fact introduce a significant difference in the bond developed by bars which are horizontal and those which are vertical, respectively, at casting?

15. For stiff mixtures vibration produces stronger and better concrete. How may this be expected to influence the bond: (a) If the concrete is vibrated during placement? (b) If vibration is delayed for several hours?

16. Bond resistance for plain bars whether mainly frictional, or adhesive, depends upon intimacy of surface contacts. To what extent may such items as richness of mixture, nature of aggregate, type of cement, use of admixtures, and the saturated or dried out condition at test, as well as period of curing, influence the resistance developed?

17. To what extent are service conditions which produce volume change alternations in the concrete likely to weaken bond because of the differential movements between concrete and steel? These conditions include alternately wet and dry, freezing and thawing and other relatively wide fluctuations in temperature.

18. After a bar has slipped in the concrete will it, after a lapse of weeks or months, resume slippage at the same or lesser load: (a) if kept moist in the interim? (b) if exposed only to a normal dry environment such as the interior air of a building?

19. A reinforced concrete beam may fail by: compression in the concrete (very rare); yield point stress in the steel; or bond and diagonal tension, combined or singly, (which of these two constitutes the primary failure is difficult to determine, since almost invariably they occur together). For current design practice, are factors of safety in bond and diagonal tension, or either of them, below those against failure by other means.

20. In the light of this survey the compressive strength of the concrete is obviously not a proper basis for the selection of design values. This implies the need for a test procedure to be used in the determination of bond strengths. What type of specimen is best suited to give the necessary information with a minimum of expense and uncertainty?

DISCUSSION OF THE QUESTIONS

The authors do not have answers for all of these questions. It is their purpose, however, to discuss each question briefly in an effort to indicate its status and in some cases to supply pertinent references. On others of the questions they can contribute, from results of their own recent tests, data that furnish answers or partial answers. Questions 1, 2, 6, 7, 8, 11, 13, 16, 18, 19, and 20 fall in this latter category.

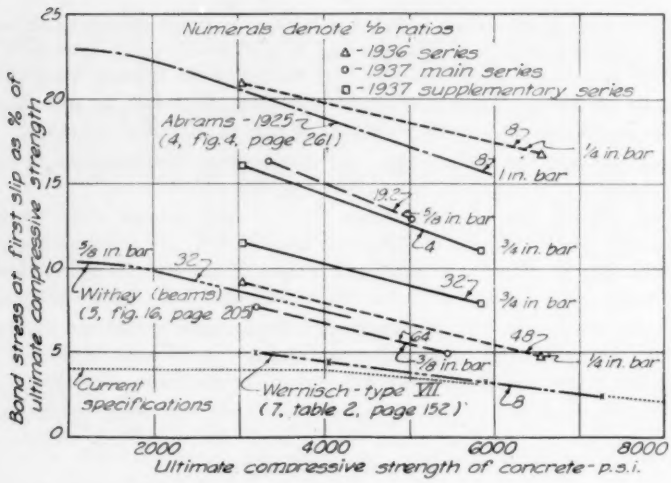

FIG. 1—BOND AT FIRST SLIP VS. COMPRESSIVE STRENGTH OF CONCRETE. ALL BARS PLAIN. ALL SPECIMENS PULLOUT EXCEPT WITHEY BEAMS

The questions will be discussed in their numerical order. References are indicated by number and refer to the list, at the end of the paper, which is selective rather than comprehensive.

1. *Strength of concrete.* For the earlier low-strength (1000 to 2000 p.s.i.) concretes such tests as those of Withey [1] [2] and Abrams [3] appeared to indicate that bond resistance varied about as the compressive strength of standard cured concrete. The more recent tests of these same investigators [4]*, discussed in [6]* and [5]*, are in agreement with recent findings by Wernisch [7]*, Davis [20]* and by the Iowa State College results of 1936 and 1937 [6]† and [8] which show that for concretes above 2000 p.s.i. the increase of bond resistance with added strength is slight. With bond resistance remaining nearly constant and compressive strength increasing, the ratio of bond resistance to compressive strength decreases rapidly as the strength of standard cured concrete increases. Fig. 1 shows the consistency of this trend according to some of the results from investigations mentioned above. The evidence seems to justify an unqualified statement that, for current concrete mixtures, bond resistance does not increase in proportion to the strength of the concrete. Furthermore it bears out the wisdom back of the usual specification provision which, while designating permissible bond stresses as a fixed percentage of the 28-day compressive strength of the concrete, also sets an upper limit of 160 p.s.i. for plain bars and 200 p.s.i. for deformed bars. That is, for concretes

*References (4) Fig. 4, p. 261; (5) Fig. 16, p. 205; (6) p. 94; (7) p. 159-161; (20) Table 1.
†Reference (6) Fig. 4, 6, 8.

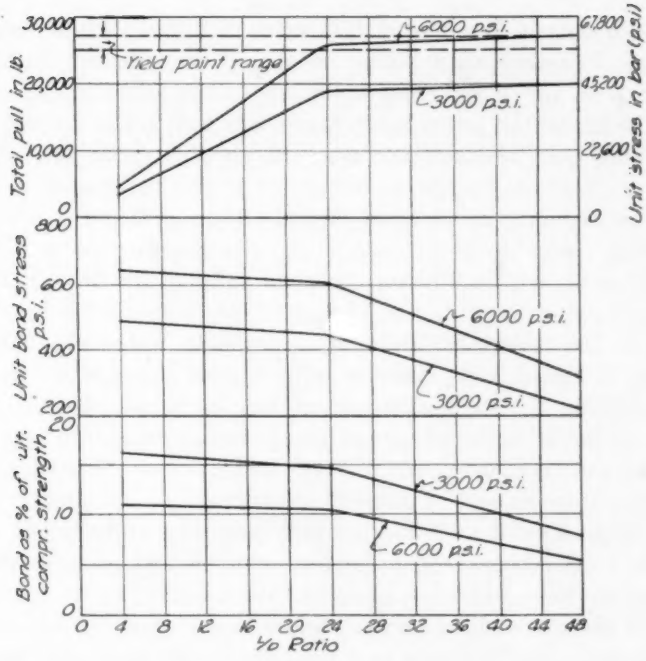

FIG. 2—EFFECT OF LENGTH OF EMBEDMENT OF PLAIN BARS ON LOADS, STRESSES, AND BOND RATIOS AT FIRST SLIP, FOR TWO STRENGTHS OF CONCRETE

Loads (top scale) are in terms of ¾ in. diameter bars. Curves are approximate mean values for Iowa State College 1936 and 1937 tests which represent results from pullout tests on both ¼ and ¾ in. bars. (References 6 and 8). Note that the two concretes reverse their relative positions in the bottom curves.

of 4000 p.s.i. or over the bond stress permitted for design is constant at these limiting values. The bottom curve of Fig. 1 shows the plotted specification requirement for plain bars. The constant 160 p.s.i. for strengths above 4000 p.s.i. for the concrete becomes a decreasing percentage as the strength of the concrete increases.

2. *Length of embedment.* The curves of Fig. 2 represent generalized mean results from the authors' 1936 and 1937 pullout tests [6] [8] on plain ¼ and ¾ in. bars, and on concretes with strengths ranging from 3000 to 6500 p.s.i. at 28 days. The scales are equally applicable to the two sizes of bar except for the upper curve for which the total pull is in terms of that on the ¾ in. bars. The original curves of which these are averages are shown on Fig. 28 of the report on the 1937 tests [8].*

*See also reference (17) Fig. 13 and 16.

Up to a length-diameter (L/D) ratio of about 24 the total bond resistance increases with added length of embedment but beyond 24 diameters, added embedment develops little added resistance. At this embedment the stress in the steel was well below its yield point for the 3000 p.s.i. concrete and near the yield point for the 6000 p.s.i. concrete. There is no visible difference in the natures of the action for the two concretes; which apparently is proof that the presence of yield point stress does not explain the phenomenon in the one case, since the stress was well below the yield point in the other. The fact that yield point stress in the steel will not account for this abrupt change in the nature of the action, seems to indicate, as does the evidence of Fig. 5, that there is sufficient slippage, from the elastic deformation in the steel, alone, to permit successive short lengths of the bar to exceed their maximum bond resistances before appreciable resistance has been developed further along the bar. That for a given bar surface there should be some limiting length of embedment beyond surface there should be some limiting length of embedment beyond which little added bond resistance can be developed, seems reasonable, and even obvious, when one considers the progressive nature of bond action as illustrated by Dunagan and Ernst [9] and by Fig. 3 and 4 of this paper. For the plain bars used in the 1936 and 1937 tests, the limiting effective embedment was about 24 diameters of bar as was noted above. These observations are in qualitative agreement with the contentions of Crum and Myers [4]. Because of the sliding resistance still present after the bond has been effectively broken, there is reason to believe that there should be a slight build-up in total (but not unit)* bond resistance, for added embedment of any amount. The available evidence does not, however, accord much relative importance to this item.

Fig. 3 illustrates, by the variations in slopes, the non-uniformity of bond stress developed along bars of relatively short embedments (L/D = 16, ¾ in. bars embedded 12 in.) for 2 strengths of concrete and for plain and deformed bars. Within limits the strains, measured at the surface of the concrete of pullout specimens, should be representative of the stresses transferred to the concrete by bond.

In contrast to Fig. 3, Fig. 4 shows a similar strain distribution diagram for plain ⅜ in. bars embedded 24 in. (L/D = 64). The bar had to slip over nearly its entire length before the end remote from the load developed any bond resistance. In studying Fig. 3 and 4 one should recognize that bond stress is present only in regions of

*Added length of embedment increases the contact area more than enough to offset such possible slight increase in total resistance from sliding friction.

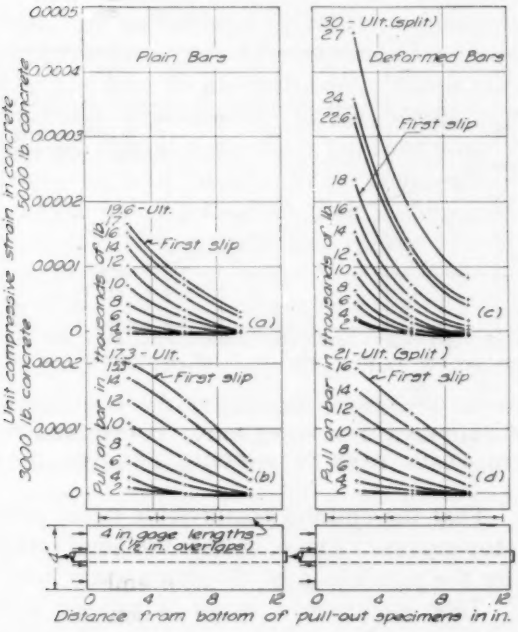

FIG. 3—DISTRIBUTION OF COMPRESSIVE STRAIN IN CONCRETE

Typical 12-in. pullout specimens with ¾ in. plain and deformed bars (L/D = 16). Plotted points are shown at centers of 4 in. gage lengths. 3000 and 5000 lb. concrete.

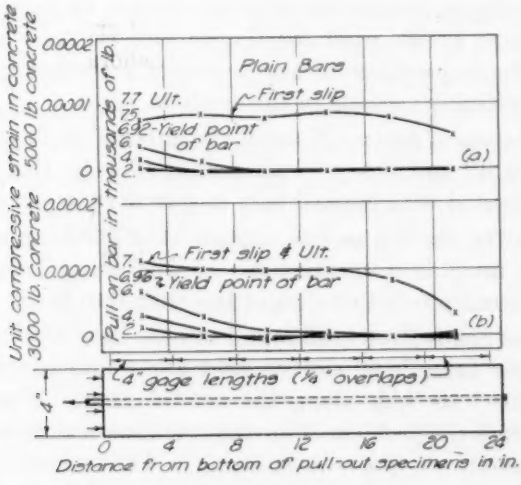

FIG. 4—DISTRIBUTION OF COMPRESSIVE STRAIN IN CONCRETE

Typical 24-in. pullout specimens with ⅜ in. plain bars (L/D = 64). Plotted points are shown at centers of 4 in. gage lengths. 3000 and 5000 lb. concrete.

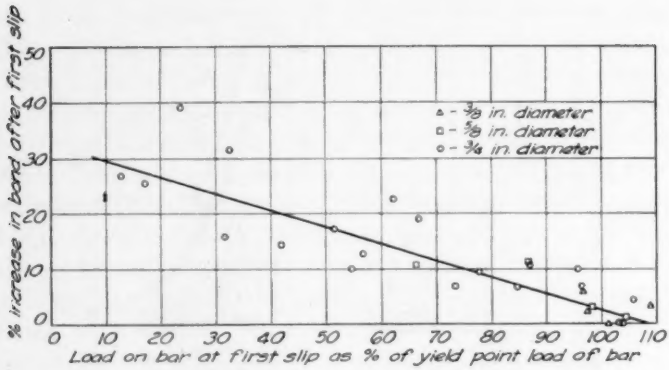
% increase in bond after first slip
50
40
30
20
10
0
0 10 20 30 40 50 60 70 80 90 100 110
Load on bar at first slip as % of yield point load of bar
△ - ⅜ in. diameter
□ - ⅝ in. diameter
○ - ¾ in. diameter

FIG. 5—EFFECT OF STRESS IN THE STEEL ON THE ADDITIONAL BOND RESISTANCE DEVELOPED AFTER FIRST SLIP. POINTS ARE FOR PULL-OUT SPECIMENS WITH PLAIN BARS AT SEVERAL LENGTHS OF EMBEDMENT

varying strain. That the uniform compressive strain in the left-hand portions of the top curves of Fig. 4, is from the bond further along the bar, is shown by the parallelism of the strain curve with the axis of the specimen. This compressive strain is present in the pullout specimen, only because of the compression transferred through the specimen to the bearing block. In a beam these strains are tensile at one face and compressive at the other. For the latter see [8]*.

Fig. 2 shows that even for low values of L/D, the unit bond resistance does not develop in proportion to added length of embedment. The same downward trends exist for the bottom curves which show, as did Fig. 1, the importance of the length of embedment or the L/D ratio as a significant variable in the study of bond.

3. *Effectiveness of hooks.* None of the authors' tests was on hooked bars. Mylrea [10] and Posey [11] are among those in this country who have experimented with hooked bars in recent years.

If bent on too short a radius enough local crushing may occur at the bend to invalidate effectiveness of anchorage beyond it, since beams have usually failed at slips of less than 0.01 in.

Long-radius bends have been found to offer little resistance, beyond that which the same length of straight bar would supply. The bar simply "snakes" its way along the curved path. When a hook is effective, there is the problem of having it enclose a sufficiently massive block of concrete to prevent splitting. The current specification requirement is to treat hooks of suitable radii (limits specified) as equivalent to an equal length of straight bar, as regards anchorage.

*Reference (8) Fig. 36-39.

Because of its varying radius of curvature a spirally bent hook (one with compound curvature) would probably offer more resistance to "snaking" than one bent to a circular arc. The authors know of no recorded tests on this type of hook but the thought of varying curvature led to the concept of a wrinkled bar, one with short waves or undulations rolled or bent into it. Some preliminary tests were not encouraging. The action resembled that of deformed bars both as regarded early initial slip and also the tendency to split or spall off the surrounding concrete.

4. *Bond in a region of compression vs. bond in a region of tension.* The extent to which the kind of stress in the concrete surrounding a bar may influence the bond stress developed is a question on which the limited evidence available from the earlier tests is inconclusive.

Withey [(2)]* produced convincing evidence that with the concrete of a pullout specimen in tension instead of under the usual compression, the bond is greatly reduced. Yet on an adjacent page [(2)]* data are shown indicating an increase in bond resistance for bars embedded in the tensile concrete of a beam. These concretes differed so greatly from those in which designers are now interested that it is best, perhaps, not to be too much concerned with trying to explain such apparent discrepancies of long standing, but rather to examine the available evidence from concrete of current strengths.

The point itself is important because it embodies one of the two primary differences between bond test results from pullouts or pushout specimens and from beams. In pullout and pushout specimens, as usually tested, the concrete is in compression, whereas in beams the steel is surrounded by tensile concrete. Consideration of this point will be resumed, therefore, under the discussion of question No. 20.

5. *Anchorage remote from region where bond resistance is needed.* Special anchorage secured by hooks, lugs, or extra lengths of embedment, often remote from regions of maximum stresses in the steel, has been acceptable under current specifications. The validity of remote anchorage is questioned only on the basis of the relatively large accumulated strain that can develop in a considerable length of bar under high stress. When one realizes that a slippage of 0.01 in. or less usually produces failure in a beam and that a 15-in. length of bar under 20,000 p.s.i. deforms this amount, it raises a question of whether or not proper interaction between concrete and steel will be secured if the anchorage is very far removed from point of need.

Remote anchorage, unless combined with pre-stressing, may be expected to produce added flexibility of member since it makes the

*Reference (2) p. 31 and 30.

reinforcement function more as a tie rod than as a bar in a homogeneous member [3]*. On the other hand the prestressing of steel and concrete, which recently has been receiving attention [12] promises to make use of remote anchorage in a proper and effective manner.

6. *Deformed vs. plain bars.* The authors' tests agree with those of Wernisch [7] and others in showing that deformed bars slip initially at bond stresses slightly higher than those for the initial slippage of plain bars. The observed increase is much below values reported by Teller [16], however. After initial slippage the resistance offered by deformed bars continues to increase until failure occurs by splitting from the wedging action of the lugs. This is at an ultimate bond stress well above that for a plain bar but much below what can be developed if splitting is prevented either by the use of spirals or by great thickness of concrete cover. Neither of these devices is practicable in ordinary construction. A beam reinforced with deformed bars may be seriously damaged prior to splitting or spalling because of the considerable accumulated slippage that occurs in bringing the lugs fully into bearing. Brown and Clark [18] found that deformed bars gave a high margin of safety under sustained loading, as did one of the authors in pullout tests through car springs [6]†.

The current types of deformed bars are far from ideal for maximum bond development as is shown by comparisons with threaded bars [3] [7] [11]. While threading is impracticable, the tests on threaded bars show that bond resistance can be greatly increased if there can be devised some fine-textured, non-wedging type of surface roughness.

In spite of their obvious drawbacks the deformed bars of current types offer some margin of added bond resistance and a considerable added margin against sudden and complete failure of beams over that offered by plain bars. The present almost universal use of deformed bars in flexural members is evidently justified.

7. *Surface coatings; rust.* Slater, Richart and Scofield [13] experimented with many types of preservative coatings including asphaltic, coal tar, ferric oxide, metallic, metal spray, red lead and phosphate, on both plain and deformed bars. All of these coatings except the phosphate decreased bond resistance, most of them by 50 percent or more for small slippages. Galvanizing gave less decrease than the other types. The nature of the action differed greatly for different coatings.

Local tests [6]‡ indicated that greasing bars greatly decreased the stress, prior to initial slippage, for both plain and deformed bars, as

*Reference (3) p. 213.
†Reference (6) p. 89.
‡Reference (6) p. 88 and 90.

was to be expected. The ultimate resistance of the deformed bars was not greatly altered by greasing, however.

Withey [2]* and Abrams [3]* both found that firm rust materially increases bond resistance which seems reasonable because it tends to produce a fine textured, non-wedging roughness. Loose rust should function more as a surface coating and should decrease rather than increase bond resistance.

8. *Surface textures.* Abrams [3]† and Wernisch [7] found that smooth or polished surfaces greatly reduced the bond resistance of plain bars, which is in agreement with the results of a few local tests on drawn wire [6]†, which gave bond strengths only slightly higher than for greased plain bars. It was found that removing the mill scale from plain hot rolled bars by over-stressing beyond the yield point, prior to embedment in pullout specimens, seemed to have little or no effect upon the bond resistance developed. The thought had been that the surface, that remained after the scaling of the oxide, might be smoother than the surface "as rolled." So far as bond is concerned, this was evidently not the case.

9. *Effect of welded intersections on bond resistance.* The authors know of no published data bearing upon this item. Undoubtedly the welded intersections function as lugs. The spacing of the welds is greater than that for the lugs on deformed bars and the effectiveness might reasonably be assumed to be below that of deformed bar lugs. For the sizes of bars normally welded into mesh, bond is not likely to be of major importance for two reasons:

(a) The superficial surface area available for bond is large in proportion to the tensile stress to be carried; (b) such reinforcement is normally used in slabs of high span-depth ratio in which shearing stresses (and, therefore, bond stresses) will always be relatively low. These two items tend to minimize the undesirable effect of the low bond resistance characteristic of a drawn surface.

10. *Two plain bars twisted together as in the reinforcement known commercially as Isteg*[1]. Twisting any form of mild steel bar raises its yield point in the outer fibers and thereby increases its range of useful stress in a reinforced concrete member. Abrams [3]‡ found that twisting did not increase the bond resistance of square bars at low slippages and from these findings it was not surprising that Wernisch [7]* should have found that two plain round bars twisted together gave about the same bond resistance as did the two bars laid adjacent to one another.

*Reference (2) p. 46-47; (3) p. 47-48.
†Reference (3) p. 48-49; (6) p. 88, 90.
[1]Isteg bars are made by twisting and pulling simultaneously and the steel is over-stressed over the entire cross section instead of in the outer fibers only, as when merely twisted.
‡Reference (3) p. 70-73; (7) p. 155.

11. *Does current testing technique give a proper indication of the bond resistance offered within a structural member?* For plain bars the ultimate bond resistance is ordinarily not far above that at initial slippage, after which the running or drag resistance drops to about 50 percent. It follows, therefore, that for loading that does not decrease as yielding occurs, a member would fail completely once its ultimate load was reached.

It was found [6]* that in accordance with expectations plain bar pullout specimens tested through springs did fail suddenly at the maximum load which for the two stronger mixtures was coincident with the load at first slip.

For the deformed bar specimens the springs did not produce failure. One specimen was kept loaded, after initial slippages, through the spring for a period of weeks. These tests illustrate the added safety factor inherent in the use of deformed bars in preference to plain.

In considering whether or not the testing machine simulates a service condition, one must recognize that in many structures the slight yielding of one member enables it to unload upon others so that it may continue to function at decreased efficiency. Certainly the pullout or beam test as normally performed does not reproduce the worst conditions of practice; testing through the springs did, however.

12. *Does bond present the same problems for steel used as compressive reinforcement as it does for tensile reinforcement?* Normally the stress, either tensile or compressive, must reach the steel through the concrete by bond. As was pointed out in the discussion of Question 5, remote anchorage may be used to replace the bond for tensile steel. In like manner adequate lateral restraint and metal plate bearings or sleeves, or welded splices at the ends may replace the need of bond for compressive steel. That this is possible was demonstrated by tests on some small columns by one of the authors [15]. Unyielding end bearings are so rare that the answer to this question is definitely yes.[1] This is especially true for compressive reinforcement in beams.

13. *Intensity of stress in the steel.* Fig. 5 seems to indicate that the tensile stress in the steel, even at a fraction of that at yield point, has a significant influence upon how much stress can be developed, in a plain bar, beyond that at which initial slip has occurred. For these tests, (Fig. 5), the maximum bond developed decreased progressively from 30 percent above that at initial slip (at 10 percent of the yield

*Reference (6) p. 90.

[1]That, as a result of the A. C. I. Column Investigation, the modular ratio n was dropped from the compressive picture, was because of interaction readjustments made necessary by shrinkage and plastic flow in the concrete and not because conditions were such as to permit each material safely to "go it alone." Plates or other unyielding end bearings, would have been necessary to produce a condition free of a need for bond.

The Poisson ratio effect is, of course, reversed, since the bar tends to swell laterally under compression. See next item.

point), down to no increase above initial slip at the yield point. Evidently the elastic-distortion (see also discussion of Question 2) and the Poisson-ratio effect (or one of them) below the yield point draws the bar away from its surrounding concrete and lowers the effectiveness of bond long before the more pronounced yield point drawing down occurs. Below the yield point the unit lateral shortening is about ¼ the unit longitudinal strain, whereas at yield point the lateral rate is doubled[1].

14. *Orientation of the steel at casting.* Water gain [14] is a very real phenomenon as is evidenced by the moisture sheen that gathers, soon after casting, at the surface of even stiff mixtures. Water gain is also apparent visually on the mortar bed below all horizontal surfaces of aggregate or steel. The bonding conditions below any horizontal bar are bad at best, and if the bar is rigidly held in place so that it cannot follow down as the plastic concrete settles, or if the mixture is at all harsh or under-tamped the bond will be still poorer. Slater, Richart and Scofield [13]* show a very bad case of broken bond involving honey-combing as well as the less spectacular water gain. While this illustrates a much worse situation than exists in the better of current construction, the fact remains that the bond for horizontally cast members such as beams will invariably be less than that for those cast vertically such as most pullout specimens. Some of the latest test data illustrating the difference are those of Davis, Brown, and Kelly [20]*. Their tests even indicate better bond for square horizontal bars than for round, presumably because about half the surface area of a round bar is subjected to water-gain effect whereas only one-fourth of the square bar faces downward. The authors know of tests in progress at other laboratories in which orientation of bars is to appear as one of the variables and which will doubtless soon supply additional numerical evaluation of this recognized variable.

In the local tests [6] [8] all specimens, both pullouts and beams, were cast vertically in order purposely to eliminate the water-gain effect in the comparisons being made between the two types. It was the desire to determine the extent to which there might be factors other than water gain to introduce differences in the bond developed in a beam and in a pullout specimen. The conclusion reached is stated in the discussion of Question No. 20.

15. *Vibration.* When vibration first entered the construction picture, there was concern in some quarters about the possibility of damaging the structure by vibrating in the vicinity of bars protruding

[1]Note that for pushout specimens the Poisson ratio effect is a swelling and observations from pushout tests supply useful additional information on this phase. (21. p. 6, 12 and 13).

*Reference (13) Fig. 51, p. 44; (20) Table III.

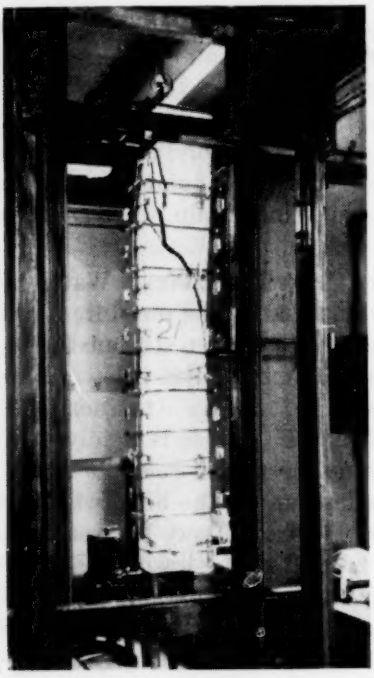

FIG. 6—TYPICAL BOND-DIAGONAL TENSION FAILURE OF A HALF-BEAM SPECIMEN

The bottom represents midspan for the corresponding simply supported full-length beam. Martens' mirrors are attached for measurements of strain in the concrete. For more complete descriptions of equipment and testing technique see Reference 8.

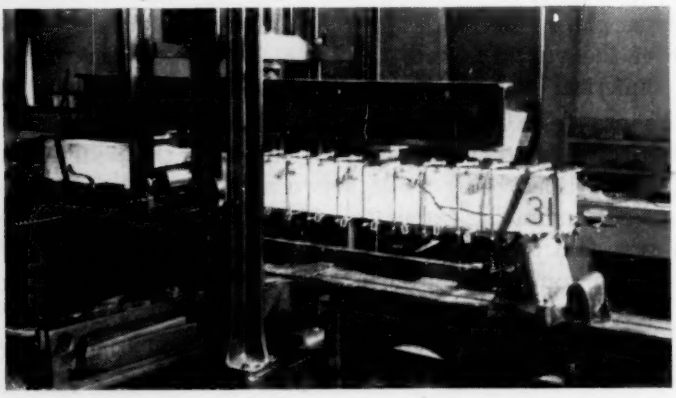

FIG. 7—A TYPICAL BOND-DIAGONAL TENSION FAILURE IN A FULL-LENGTH BEAM TESTED HORIZONTALLY

See also Fig. 6 and Reference 8.

from concrete previously placed. Withey [5] dispels that fear as regards hardened concrete and Davis, Brown and Kelly [20]* show that vibration is very beneficial to bond if done at any time up to nine hours after placement. Attaching the vibrator directly to the bar gave the best results. It is possible that the remixing action secured by delayed vibration may be the best possible antidote for the objectionable water-gain effects beneath horizontal bars. The delayed vibration tests reported were on vertical bars and evidence on this point is not, therefore, available.

16 and 17. *Contemporary tests.* A contemporary paper by Davis, Brown and Kelly [20] contributes new and timely information on a variety of hitherto unexplored topics so far as bond is concerned. Of special significance are their comparisons of different cements, (see also Glanville [21]†) two admixtures (T. D. A. and puzzolan) and volume change effects (freezing and thawing, and wetting and drying). Their contributions on vibration and orientation of bars have already been mentioned. To abstract these readily available results here would be superfluous.

18. *Autogenous healing in bond.* Abrams [3]‡ noted the phenomenon of autogenous healing as applied to bond and the same was noted in the pullout tests of 1936 [6] which were retested in 1937 at the age of a year. Such action, while not usually to be depended upon, does give an added factor of safety when there has been an over-strain followed by favorable conditions for resumed curing or healing action. There is no recovery under dry conditions.

19. *Factors of safety in bond.* The authors' tests [8] have produced evidence that on the basis of current specifications factors of safety against bond and diagonal tension failure or either of these factors are lower than those against failure by compression in the concrete (for such failures are very rare). The factor of safety is usually above that against failure by yield point stress in the steel, however. While this indication is reassuring it is not a cause for complacancy regarding the many factors that still remain undetermined or misunderstood, relative to bond, and how to develop bond resistance most effectively. Failures by bond and diagonal tension, or either of them, are always sudden and complete as illustrated by Fig. 6 and Fig. 7 in contrast with the tensile failure of Fig. 8.

Results from 32 beam tests [8] gave factors of safety on the basis of current design practice as follows:

*Reference (20) Tables IV, V, VI.
†Reference (21) p. 6.
‡Reference (3) p. 215.

RANGES IN FACTORS OF SAFETY AT FIRST SLIP

Concrete Strength	Tension in Steel	Compression in Concrete	Bond	Diagonal Tension
3,000	2.45—3.74	4.12	2.27—3.87	2.79—4.56
5,000	2.43—3.60	No failure	2.96—4.27	2.84—5.16

These beams were vertically cast. Had they been cast horizontally, the factors of safety in bond would be somewhat lower, whereas the other factors of safety should be unaffected.

20. *What constitutes a suitable specimen for bond tests?* At the outset one must recognize that a suitable specimen for any test may give relative rather than absolute indications. The need for bond occurs most frequently in beams but the casting and testing of reinforced beams is laborious and costly and not at all suitable for standardized testing. As a criterion of bond behavior either the pullout or the push-out test is acceptable if the results obtained parallel those secured from beam tests.

The authors' tests [17]* have shown that when pullout and beam specimens were both cast vertically, they gave identical bond strengths for equal length-diameter (L/D) ratios. Since in the beams tensile concrete surrounded the bars and in the pullout specimens the concrete was in compression, the old question of compressive vs. tensile concrete appears definitely to be answered in favor of the contention of Abrams [3]* and in opposition to that of Withey [2]*

As emphasized previously, the beam bonds may be expected to be less than pullout bonds, but all available comparisons show excellent correlations between the two. Wernisch's tests [17]† show good parallelism as do those of Withey [2]† and Abrams [3]†. Thus there apparently is agreement in the indications from all of the comparative pullout versus beam tests that parallelism of behavior does exist. There seems, therefore, to be no valid question regarding the dependability of the pullout specimen if provision be made to control or evaluate that other important and heretofore ignored variable, "The L/D ratio or length of embedment."

Within limits the diameter of the enclosing concrete block is relatively unimportant [21]‡ but the length of the block** needs to be varied to suit the diameter of bar being tested. The selection of a suitable value of the L/D ratio for such tests calls for preliminary study of several of the factors involved. Some value of L/D between 10 and

*Reference (17) Fig. 16, p. 164-7; (3) p. 104, 214; (2) p. 33, 46.
†Reference (17) Fig. 14, p. 164-6; (2) p. 33; (3) p. 148, 219-220.
‡Reference (21) p. 13.
**Glanville (21) believes that it is length of embedment (L) rather than relative length of embedment (L/D) which is significant. For tests on one-size bars the two criteria are the same.

FIG. 8—A TYPICAL FAILURE BY TENSION IN THE STEEL OF A HALF-BEAM SPECIMEN
See also Fig. 6 and Reference 8.

16 might be reasonable. Abrams [2]* recommends 8 diameters. What the authors deem an undesirable alternative would be to use equal lengths of embedment for the several sizes of bar and to correct with some predetermined factor for the variations in L/D ratio due to differences in the bar diameter, D. (unless Glanville's conclusion proves to be correct; see footnote p. 16).

Several of the current investigations are being based entirely or in part upon pushout rather than pullout tests. These tests are even simpler to conduct than the pullout tests and it is possible that the adoption of the pushout test will be the better solution. The L/D ratio would need the same special consideration as for the pullout type. The concrete around the steel is under compression for both pullout and pushout but for the pushout the steel is under compression, where it enters the concrete, and instead of tending to be drawn down, it tends to expand against the concrete. This difference increases

*Reference (2) p. 220.

bond values for the pushout test but there is no reason for not expecting reasonably satisfactory correlation between the two.[21]*

The answer then seems to be that the needed bond test specimen should be either a pullout or a pushout specimen with suitable provision for eliminating or evaluating the length or the length-diameter (L/D) variable.

COMMENTS

In conversation F. R. McMillan has pointed out and emphasized that what we have always termed "first slip" is in reality the "last slip," since the bar has then slipped over its entire length. Things first begin to happen at the point where the bar enters the concrete at the loaded end. Movements here are difficult to measure but some of the current or future investigations may be expected to supply the needed factual basis for further analysis.

The bar of a long pullout specimen will certainly have moved much farther with respect to the concrete at point of entry prior to the so-called first slip, than will the bar of a short pullout specimen. Glanville [21]† doesn't approve of "the first slip" criterion so much used by American investigators.

CLOSING REMARKS

This necessarily sketchy resumé reemphasizes the extent to which the early works of Withey and Abrams have controlled our thinking on the subject of bond right down to the present day. There are now, however, some concepts that need to be modified and the more important of these are:

For concretes stronger than 2000 p.s.i. (at 28 days) bond resistance does not increase at nearly the same rate as does compressive strength. Bond design stresses should be based upon bond tests rather than upon compressive strengths.

Bond resistance does not increase directly as the added length of embedment and for any type and size of bar the unit effectiveness in bond is a function of the embedment length at which it is to be used.

In addition to the foregoing, there seems now to be conclusive evidence that the pullout and probably the pushout specimen can be made to give dependable indications relative to bond behavior in beams. This is one of the questions that has been disputed down to the present time.

The use of deformed bars is sound practice, although better types of gripping surface should be developed.

*Reference (21) p. 6, 12, 13.
†Reference (21) p. 23.

Because the authors have taken it upon themselves to discuss rather dogmatically the problems of bond as they visualize them in the light of their own tests and supplementary study, they have deemed it essential to supply more of specific reference material than might otherwise have been necessary or desirable. In a field so large the questioning reader needs to be able to contact the sources easily. That is the justification offered for the degree of documentation accorded in this treatment.

ACKNOWLEDGMENTS

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Discussion of the foregoing paper will be welcome if received in triplicate by the Secretary of the Institute by December 1, 1938. For such discussion as may develop readers are referred to the JOURNAL *for February 1939.*

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Discussion of a paper by Gilkey, Chamberlin and Beal:

The Bond Between Concrete and Steel*

BY WARREN RAEDER, GEORGE ROBERT WERNISCH
AND CLOSURE BY THE AUTHORS

BY WARREN RAEDER†

In their excellent and valuable summary of the major problems of bond under 20 headings the authors state in answer to query No. 9 ("How much is provided in added effectiveness of bond resistance by the welded intersections of a mesh or mat?") that they "know of no published data bearing upon this item. In 1932-3 the writer in conjunction with Charles Neel, at the time a candidate for the M. S. degree at the University of Colorado, made a preliminary study of bond between concrete and welded wire reinforcement. The following discussion, necessarily brief, arises from that study.

Welded wire reinforcement is used most commonly for floor slabs and roof slabs and is made of cold drawn wire of relatively small diameter for longitudinal reinforcement. It has cross wires of smaller gage welded about every 12 to 16 in. which act as temperature steel and as spacers for the main steel. The cold drawn wire used in these experiments had an ultimate strength of about 79,000 p.s.i. and a limit of proportionally of about 85 per cent of the ultimate.

From the view-point of bond it should be noted that cold drawn wire has an extremely smooth surface which reduces the adhesion to concrete. On the other hand the cross-wires presumably contribute to bond resistance by furnishing an excellent mechanical anchorage. The data show this to be the case.

Tests on the shearing strength of the welds on commercial fabric gave the results (average of three specimens) tabulated below:

*JOURNAL, Amer. Concrete Inst., Sept. 1938; *Proceedings* Vol. 35, p. 1.
†Assoc. Prof. of Civil Engineering, University of Colo.

Longitudinal Wire			Cross-Wire Sizes	Shearing Strength	% of Tensile Main Wires
Size	Diam.	Area			
7 ga.	.177"	.0246 sq. in.	11 ga.	1260 lb.	62
4 ga	.225	.0397 sq. in.	9 ga.	2705 lb.	88
0 ga.	.307	.0738 sq. in.	6 ga.	2998 lb.	52

It should be noted that the strength of two welds is greater than the ultimate strength of the reinforcement.

Tests were made on three types of steel and concrete specimens: pull-out, beams, and slabs. Concrete of only one mix was used: nominal $f_c' = 3000$, actually 2 per cent less than that. Maximum size of aggregate was $\frac{3}{4}$ in. All pull-out cylinders were of 3 or 6 in. diameter, embedment nominally 3 in.

Pull-out tests were made on plain cold drawn wire of two gages, No. 7 and No. 4; $\frac{1}{4}$ in. round plain ordinary reinforcing bars; single units of mesh consisting of one main wire with one cross wire; and units of two longitudinal wires with a cross-wire welded across both. All cross-wires cantilevered out from the main wire 1 in. or 2 in. respectively on each side, no difference in results being observed between the two. Main bars extended $\frac{1}{2}$ in. beyond the top of the specimens giving bearing for Ames dials used to measure slip. All specimens were cast with the same orientation as tested, were cured in water and, unless otherwise noted, were saturated at test. Results given in Table 1 (a partial summary) are averages of three specimens.

Pull-out specimens were tested in a 20,000 lb. hand operated Olsen testing machine, the specimen supported on a spherical bearing block. An Ames dial reading to .001 in., held by a yoke which was clamped around the specimen, measured the slip of the bar. Because the plain unfabricated wire tended to pull out in jerks rather than uniformly, Table 1 shows the load at actual first slip, the latter varying for different specimens. Slips less than .001 in. were estimated.

Typical action of plain wire (without cross-wires) in these pull-out specimens was as follows. The load (lines 1 and 2, Table 1) increased close to its maximum of about 300 p.s.i. or $.10f_c'$ without any slip. Then first slip of several divisions on the .001 dial occurred and the bar pulled out in increments of about 1 division and at a load only slightly less than the maximum. Apparently the very smooth surface of the cold-drawn wire allowed a larger first slip than the less smooth surface of hot-rolled bars. As will be noted later, the action in beams was similar. For plain, unfabricated wires the beams failed at first slip. Of course it must be remembered that, as pointed out by the authors, it does not take much slip of reinforcement in a beam to cause its failure.

TABLE 1—SUMMARY OF PULL-OUT TESTS

(Each line represents the average of three specimens)

Line	Gage	Bar Diam. In.	L/D	Age Days	First Slip			Max. Load			Remarks
					Lb.	p.s.i.	Slip, In.	Lb.	p.s.i.	Slip, In.	
1	7	.177	17	28	508	292	.008	Same as at 1st slip			
2	4	.225	13	28	672	288	.004	Same as at 1st slip			
3	...	¼	12	28	1310	513	.0002	1550	608	.003	
4	4	.225	13	28	60	28	.....	60	28		Greased.
5	4 long. 9 x-wire	.225	13	28	285	...	.0002	2325	...	.041	Weld snapped at ultimate.
6	"	.225	13	28 wet 7 dry	650	...	.0003	2725		.047	Weld snapped.
7	"	.225	13	28	203		.0004	1500		.120	Long. bar greased. See text.
8	"	.225	13	28	1355		.001	5467		.045	Two long. wire See text.
9	"	.225	13	28	680		.001	4500		.070	Greased.
10	0 long. 6 x-wire	.307	10	28	630		.0005	4500		.065	
11	"	.307	10	28	65		.0005	3225		.090	Greased.

TABLE 2—BEAM SPECIMENS

Type of Steel	b ins.	d ins.	L ins.	a[1] ins.	% of Steel	First Slip (.0001")				Maximum Load				Type of Fail
						Load lbs.	Mom in.-lbs.	S p.s.i.	U	Load lbs.	Mom in.-lbs.	S p.s.i.	U	
4 ga.	3	3	24	6	.54	1450	4350	43,500	446	1460	4380	43,800	450	bond
(0.225)	3	3	24	4	.54	2320	4640	46,400	656	2320	4640	46,400	656	bond
(0.225)	3	3	24	4	.54	2250	4500	45,000	635	2320	4640	46,400	656	bond
¼-in.	3	3	24	4	.65	2000	4000	32,600	553	2580	5160	42,100	713	bond
straight	3	3	24	4	.65	2700	5400	44,000	745	2800	5600	45,600	772	bond
hooked	3	3	28½	4	.65	....	....	.....	....	4600	9200	75,000	1170	ten.
hooked	3	3	28½	4	.65	....	....	.....	....	4250	8500	69,500	1080	ten.
BF49*	3	3	24	4	.54	1940	3880	38,800	548	3960	7920	79,200	1120	ten.
	3	3	24	4	.54	1900	3800	38,000	536	4920	9840	98,400	1390	ten.
	3	3	24	4	.54	1500	3000	30,000	424	3700	7400	74,000	1045	ten.
greased	3	3	24	4	.54	1360	2720	27,200	384	3630	7260	72,600	1028	ten.
greased	3	3	24	4	.54	1100	2200	22,000	310	3740	7480	74,800	1055	ten.
0 ga.	3	4¼	24	6	.66	2300	6900	26,800	348	3650	10950	42,500	552	bond
0 ga.	3	4¼	24	6	.66	2200	6660	26,000	336	4500	13500	52,500	680	bond
0 ga.	3	4¼	24	4	.66	4000	8000	31,000	605	6300	12600	49,000	955	bond
0 ga.	3	4¼	24	4	.66	3800	7600	29,600	575	6400	12800	49,800	970	bond
BF49*	8	3	46	19	.40	1180	11200	56,000	167	1250	11900	59,500	177	ten.
	8	3	46	10	.40	1720	8600	43,000	244	2920	14600	73,000	415	ten.
greased	8	3	46	19	.40	680	6450	32,250	80	1580	15000	75,000	224	ten.
greased	8	3	46	10	.40	1200	6000	30,000	170	1600	8000	40,000	228	bond
BH49*	6	3	46	19	.53	1100	10500	52,500	313	1700	16200	81,000	483	bond
	6	3	46	23	.53	1350	15500	77,500	384	1600	18400	92,000	455	ten.
greased	6	3	46	23	.53	700	8050	40,250	199	1500	17300	86,500	425	bond
greased	6	3	43	21½	.53	500	5370	26,850	142	1040	11200	56,000	296	bond

[1] "a" is the distance from the support to the nearer of the symmetrical loads.

*The first letter denotes spacing of longitudinal wires, A, 2 in., B, 4 in.; the second letter the corresponding spacing of the cross wires; the numerals the gage of the main and of the cross wires.

Plain ½-in. hot rolled reinforcing bars (line 3) developed 513 p.s.i. at first slip of .0002 in. and a maximum of 20 per cent additional at .003 in. slip.

The anchorage afforded by the cross-wire welded to the main wire seemed to smooth out the jerky motion noted in plain material. For 4-9 mesh (line 5) first slip occurred at .0002-in. at a total load of 285 lb. as against 508 lb. in the case of the plain wires. Final failure caused by shearing of the weld took place at a load of 2375 lb. and a slip of .04 in. In estimating the additional strength of the welded cross-bar one must recall that in the commercial fabric the cross-wires are spaced up to 16 in., whereas in the pull-out specimens the length of main wire in bond was only three inches. For this reason Table 1 contains no "p.s.i." value for the fabricated specimens.

An attempt was made (line 7) to test the strength of the weld by greasing and wrapping the longitudinal wire with thin, oiled paper. In testing however, the slipping proved to be so great that the welded joint was not subjected to true shear as had been planned. In fact failure in each of three specimens was by rupture of the cross-wire on one or both sides of the weld which, itself, remained intact. The displacement of the welded spot and therefore, the deflection of the cross-wire amounted to as much as ¼-in. at failure. This situation probably accounted for the discrepancy between the 1500 lb. ultimate load for the pull-out specimens and the average of 2705 lb. for the weld noted above. As will be noted results of the other greased specimens were more satisfactory.

For specimens with two longitudinal wires with one cross-wire between them (line 8) and cantilevering out 1 in. on each side no difficulty was encountered in procuring equal or near-equal amounts of slip on the two wires. Although the welds did not break simultaneously, both were probably stressed close to the breaking point when the first failed. In these specimens "first slip" of .001 in. is not quite the whole story because no attempt was made to estimate smaller amounts in the specimens with two dials. The wires did not pull out in jerks as did the plain unfabricated specimens.

The greased specimens (line 9) failed in this instance by shear, the ultimate strength of 4500 lb. comparing not unfavorably with the 2705 lb. for one weld already noted.

In the slightly heavier fabric, 0-6 (line 10) the longitudinal wire had an area 1.86 times that of the No. 4 wire of the 4-9 style. Ultimate strength of the larger fabric was 1.93 times that of the smaller which indicates that the proportions of the two sizes of mesh are consistent. The greased O-6 specimen (line 11) showed an ultimate strength of 3225 lb. against 2998 lb. recorded in the tests of welds not embedded in concrete.

Beams. The beams tested ranged from 3 to 8 in. wide and 26 to 48 in. long and were designed to fail in most instances by bond. Depths were mostly 3 in. overall, 2½ in. effective. Reinforcement varied (see Table 2) from plain wire and plain ¼-in. diam. bars to fabricated "mesh," AF49, BH49. In this designation the first letter represents the spacing of the longitudinal wires, in multiples of 2 in., A standing for 2 in., B for 4 in., etc.; the second letter the corresponding spacing of the cross-wires; the numerals represent the gage of the main and of the cross-wires.

All beams were tested wet at 28 days. Slips of the ends of the main bars was measured by .0001″ "Last Word" dials supported on wires embedded in the concrete not closer than 1 in. to the main steel.

Loading was two-point symmetrical, the "a" distance (from support to nearer load) being so close in most instances as to cause bond failures. In some cases "a"

was made large to induce larger stresses in the steel and so test the anchorage qualities of the cross-wires. Most specimens failed in bond, some in tension or compression.

Bond value was given were computed from $U = \frac{V}{\Sigma o jd}$, j being found from the transformed cross-section. The same value of j was used in finding S, the stress in the steel, and this probably accounts for the high value given in some specimens at ultimate load.

Slabs. Because the beams contained for the most part only one bar, slabs four times as wide and containing four bars were made and tested to determine whether there was any difference between their behavior and that of beams. Approximately the same ultimate stresses were found to develop and failures were the same as those of the corresponding beams. Justification for applying to slabs conclusions drawn from tests on beams was thus established.

Deflections taken at the center of the span indicated that the stiffest slabs were those reinforced with wire mesh, the most flexible those with plain cold-drawn wire. Those with ¼-in. hot rolled bars lay between these extremes.

Conclusions

1. The cold-drawn wire, the constituent of the wire mesh reinforcement, had an ultimate strength of about 78400 p.s.i. and a proportional elastic limit of 85 per cent of the ultimate.

2. Shearing strength of the welds was 1260 lb. for 7-11 mesh, 2705 lb. for 4-9 mesh, 2998 lb. for 0-6 mesh. The strength of two welds in shear was greater than the ultimate strength of the longitudinal wire.

3. The maximum bond resistance of plain unfabricated cold-drawn wire, is developed at first slip. For 3000 p.s.i. concrete, 28 days old and tested with L/D = 12, the unit bond stress was about .10 f_c'. Corresponding values for ordinary hot rolled ¼ in. bars were .17 f_c' at first slip and .20 f_c' at maximum.

4. For specimens with a cross-wire, first slip was much less than for plain wires and at an appreciably lower load.

5. The weld on the cross-wire of 4-9 material failed at a total slip of about .04 in. For the conditions of these tests the ultimate load was about 3½ times that of specimens made with unfabricated material.

6. Beams reinforced with plain cold-drawn wire failed suddenly and at first slip at a bond of 580 p.s.i. (.19 f_c'). Beams with ¼ in. round reinforcing bars recorded a first slip of .0001 in or an estimated fraction thereof at 650 p.s.i. (.22 f_c') and a stress at failure of 792 p.s.i. (.26 f_c').

7. The cross-wires on the welded specimens (BF49) tended to equalize the slip of both ends of the specimens but did not increase the load at first slip.

8. The anchorage qualities of wire mesh for a beam with only one cross wire at each end of the beam and one at the center is of a magnitude comparable to that of ¼ in. bars hooked at the ends. With failures in these specimens due to compression (or tension) the computed bond amounted to 1185 p.s.i. for the mesh and 1125 p.s.i. for the hooked bars.

9. At ultimate loads bond acts as well as anchorage of the cross-wires although its contribution is relatively small.

10. Greasing the longitudinal bars reduces the load at which first slip occurred.

11. The behavior of slabs was the same as that of the narrower beams similarly reinforced.

12. Slabs reinforced with BF49 mesh deflected less than those reinforced with ¼ in. reinforcing bars.

Acknowledgment

These experiments were carried out in the concrete laboratory of the University of Colorado, Department of Civil Engineering, of which Prof. C. L. Eckel is head.

BY GEORGE ROBERT WERNISCH*

This excellent paper and the author's reference No. 8 indicate that only a limited length of bar is really effective in resisting bond stresses at any time, and that increasing embedment beyond this length decreases the unit bond stress, but not the *total* bond resistance appreciably. This may be verified in the authors' Fig. 2.

It is important and assuring to know that the total bond resistance remains approximately constant for embedment-diameter ratios in excess of 24, regardless of the length of embedment; for generally more than 24 diameters embedment beyond points of maximum bond stresses are necessary to tie in the structure, take care of non-uniform loading, etc.

General bond design practice has been based on the theory that increased embedment provided correspondingly greater bond resistance; and the effectiveness of embedment anchorage far from the point of maximum stress has been taken for granted.

In practice sufficient bond resistance is obtained by hooking, in some few cases by special anchorage such as enlarged end sections, cast iron sleeves, etc. and by increasing the length of embedment.

Forssell (*Betong* No. 1, p. 31-78, 1937) found that hooking did not give very satisfactory results, but that special anchorage as mentioned

*Ceco Steel Prod. Corp., Chicago, Ill.

heretofore was very effective. The authors' data indicate that increasing the length of embedment does not increase total bond resistance for L/D ratios in excess of 24.

Hooking and remote anchorage being either ineffective or prohibitively costly (in cases of special types of anchorage) in increasing bond resistance, we must develop new and inexpensive methods of obtaining sufficient bond development at points of maximum stress. It is therefore desirable and practical to incorporate more bond development into a new type of deformed bar, rather than to rely on ineffective and expensive bond development as discussed previously. The writer is convinced from observation and experiment that present commercial bars, although superior to plain bars, do not develop sufficient resistance at points of maximum stress. Also the variation in effectiveness of different types of commercial deformed bars is entirely too great; there appears to be need for more uniformity in the deformations used by steel producers. Two bars with different deformations may vary as much as 40 per cent in pullout resistance when embedded in concrete of equal strengths. A diagonal type of deformation appears to be superior to the transverse and longitudinal types, with the added advantage that the steel in the diagonal deformations aids in resisting tensile stresses because bars containing this type are of constant section throughout. Though threaded bars are vastly superior to diagonal deformed bars, the writer agrees with the authors that they are now commercially impractical. Practical variations thereof would improve bond resistance of bars considerably.

Almost every steel corporation rolls a different type of deformed bar. Several years ago there were over 20 types on the market. The type apparently serves primarily to identify the product. The industry would do well to study various deformations and choose a standard to embody low rolling costs and high bond efficiency.

AUTHOR'S CLOSURE

The authors feel that both of the discussions offered constitute desirable and appropriate supplements to the paper. The wording of Item 9 (page 11 of the paper), to which Porfessor Raeder alludes, was selected advisedly. The authors could truthfully have stated that they "knew of some *unpublished* data probably bearing upon this item." Having known as they did of Professor Raeder's work, they felt and still feel that his discussion might well have been offered as a paper in its own right. The author's believe that their paper has at least one accomplishment to its credit, namely that of having supplied

the stimulus which at last makes Raeder's findings a matter of published record.

The function of the authors' paper was to summarize the general subject of bond in the light of present knowledge. They do not see that any of Raeder's results contradict statements made in the paper but they do add to the knowledge available. This amplified case for the welded mesh can now be presented about as follows:

(a) Where there is need for added bond resistance by a welded mesh, the cross-wire may be depended upon to function essentially as do the lugs of a deformed bar.

(b) Because of the smallness of the wires which are welded into mesh, splitting at the intersection will rarely be the problem that it is at the lugs of the sizes of rolled deformed bars normally used in beams and slabs.

(c) Only that portion of the cross wire immediately adjacent to the main wire is effective for lug action because of the very limited give that can occur prior to too great damage to a flexural member.

(d) The welded intersections on the bars of Raeder's tests were more closely spaced than those of commercial mats and the effectiveness of the intersections should be correspondingly more pronounced for Raeder's samples.

(e) The welded lugs have little influence upon the load and stress at which "first slip" occurs and like any other type of deformation their primary function is that of providing a reserve toughness or resistance to collapse after enough slippage has occurred to bring the lug or intersection into bearing. The fact that two intersections will develop the full strength of the main wire is of significance only because of the insurance against sudden collapse. It offers no insurance against distress of an order that would be structurally disqualifying.

(f) The Raeder tests seemed to provide little evidence of splitting or spalling from the lug action of the cross-wires. This lack of spalling is a reasonable indication because of the smallness of the wires used in mesh. It constitutes an added benign factor in its bearing upon the ultimate or late-stage toughness mentioned in (e).

(g) The drawn wire of which a welded mesh is normally made has, as both Raeder's and other tests have shown, a smoother surface than one which is rolled and slippage occurs therefore at a lower intensity of bond stress for the wire.

(h) The small sizes used for mesh have a relatively high specific surface in comparison with the specific surface of the bars of larger

diameters; which fact, as has been pointed out in the paper, tends to offset the undesirable effect of added smoothness of surface.

(i) A drawn steel wire has, by virtue of the cold working from drawing, a high yield point which is, to an increasing extent, coming to be recognized as an asset in a reinforcing bar; since major cracking of concrete flexural members is inevitable for stresses above the proportional limit of the steel.

(j) After one has duly weighed the pros and cons of what the authors believe to be the salient facts which bear upon welded wire mesh in its relationship to bond, he needs to detach himself sufficiently to recognize that only occasionally will bond be an important consideration for a mesh anyway, since mesh is invariably used in the construction of thin slabs in which shears and therefore bond stresses will generally be relatively low.

Mr. Wernisch calls attention to some important factors relative to deformed bars. The statement made in his opening paragraph might be open to slight misinterpretation. To delete the word "appreciably" would accomplish the needed change, since the authors found *no decrease* in *total bond resistance* with increased length of embedment, although above about 24 diameters, the maximum resistance that could be developed by a plain bar remained virtually constant. As the authors have pointed out more fully in a subsequent paper now in process of publication[1] there should be some increase in the total resistance developed by plain bars as the length of embedment is increased because each added inch of length does add an increment to the total "drag resistance" which remains after any part of a bar has passed its own ultimate resistance. This added drag resistance supplements the resistance offered by that portion of the bar which has not developed its maximum.

In considering an ideal bar for bond one must give weight to several factors not all of which are compatible one with another. For example the threaded bar would be ideal for bond, as has been demonstrated, but even if the threaded bar were a feasible thing to manufacture, the bases of the threads would be objectionable as "stress raisers" even under static loading and also in bending. Moreover during any form of exposed storage, they would doubtless be vulnerable to corrosion.

The writer's tests[2] with a variety of deformed bar types make him something of an authority in that field. Apparently the type of bar

[1] "The Distribution of Strain in the Concrete of Pullout Specimens", Proc. Highway Research Board, Vol. 18, (1938).
[2] JOURNAL, Amer. Concrete Inst., Nov.-Dec. 1937; *Proceedings* Vol. 34, p. 145.

to which he refers would be such as III and IV of his tests. Such bars should not seemingly be objectionable from the standpoint of added tendency to split the concrete. On the other hand the old Havermeyer bars, some of which were similar to Mr. Wernisch's bars V and VI, are probably objectionable from the standpoint of wedging, or splitting action although in some respects they fit the wording of his specification.

As the authors have stated before, the present evidence seems to indicate that the best bar will be that which comes nearest to having a surface of fine textured roughness. This is receiving increased recognition and Mr. Wernisch becomes one more to point out or to imply that the need for a bar with such a surface is both real and urgent.

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Concrete Wearing Surfaces For Floors*

Report of Committee 804

J. FRUCHTBAUM, CHAIRMAN

SYNOPSIS

Assigned to present data and recommendations in reference to concrete floor surfaces for hard wear, Committee 804 supplements with a record of new accelerated tests, data previously presented to the Institute. The test data, plus experience with thousands of square yards of floor surface now giving good service, suggest a specification outline for materials and workmanship both held within narrow limits. The specification is a departure from usual practice and is presented now for criticism and to develop suggestions for what the committee may later propose as specifications and recommended practice for adoption as an Institute Standard. The committee suggests as a basis for further tests and field experiences, a 1:1¼:2 mix; 4 gal. of water per bag of cement for power floating—5 gal. for hand floating and the use of silica, traprock or granite or other material of equivalent hardness.

INTRODUCTION

In its study of factors of materials and workmanship for wearing surfaces for concrete floors, the Committee has taken advantage of the work reported briefly by the late L. F. Fairchild in his paper "Concrete in Factory Construction," (JOURNAL, Am. Concrete Inst. Nov.-Dec. 1934, *Proceedings* Vol. 31, p. 149.)

Mr. Fairchild's tests were conducted so that, by the process of elimination, there was always a sample of the best previous finish to compare with the new samples. There were four series of these tests using such aggregates as ordinary sand, iron ore tailings, crushed quartz and pure silica sand, with varying fineness moduli and varying strengths. The results indicated the best combination to be a fineness modulus of approximately 3.93 and a crushing strength of 5500 p.s.i. using the hardest available aggregate.

*Received by the Institute January 17, 1938.

It was thought well to check some of the results obtained in 1933 from Mr. Fairchild's tests and to consider the cement factor further in reference to a number of cements now being marketed for special purposes, to determine whether two common types of cement offer advantage over standard portland, in resistance to wear and to acid action.

The amount and grading of aggregate were varied in some of the samples, as this appears to be a major factor in wear resistance.

The committee is greatly indebted to the Eastman Kodak Co. for permitting the use of its test data. The report on the tests which follows, is the work of Morgan B. Klock, (member of this committee) of the Eastman company.

WEAR TESTS

Preparation of Test Floor

A base slab was first cast on the ground and allowed to harden. Just before the 1 inch thick samples of finish were applied, a 1:1 grout was well scrubbed over the surface with a wire brush. The finishes were mixed to such consistency that the material could be packed into a ball with the hands and scarcely leave any moisture on them. It was then tamped, screeded, and floated immediately with a power float. Each sector was hand trowelled three times. The test sections were cured under wet burlap, the standard portland cement mixtures for one week, and the high early for two days.

Testing Device

The device used for wear tests* consists of horizontal arms attached to a vertical shaft which was revolved. Two loaded trucks were attached to the horizontal arms in such a manner that the wheels of both trucks traveled in the same track as they ran in a circular path over the finish to be tested. The test samples were laid in sectors to form a circular area. Each truck had a total load of 315 lb. and was provided with three swivel casters 3 in. in diameter by 1½ in. wide.

The apparatus was rotated at about 8 r.p.m. until a definite comparison could be made. The depth of wear was measured with a bridge micrometer in the outer track, the radius of which was approximately 5 ft. 6 in.

Samples of Floor Finish

Sample wearing courses were made from portland and High Alumina cements.

The mixes used in the sectors of this floor were as follows:

*See illustration p. 158, Mr. Fairchild's paper, JOURNAL Concrete Am. Inst., Nov.-Dec., 1934, *Proceedings* Vol. 31.

1. Portland cement *A* and silica aggregate, 1 to 2.67 by volume; fineness modulus of aggregate 4.1; 3.2 gal water per bag.

2. Portland cement *A* and silica aggregate, 1 to 3.1 by volume; fineness modulus of aggregate 4.1; 3.12 gal. water per bag. This is the mix selected in 1933, after the first series of tests.

3. Portland cement *A* and silica aggregate, 1 to 2.32 by volume; aggregate: 67 per cent passing ⅜ in. sieve and retained on No. 4, 33 per cent graded from 4 per cent retained on No. 30 sieve to 6 per cent passing the 100 sieve; water 2.85 gal. per bag. This was intended to duplicate a mix for miniature concrete used for testing cement. Actually it is much richer than it should be.

4. Portland cement *A* and silica aggregate, 1 to 2.67 by volume; fineness modulus 3.68; water, 3.28 gal. per bag.

Samples 1 to 4 were 33 days old at the beginning of the test, after moist curing for one week.

5. High alumina cement and silica aggregate, 1 to 2.67 by volume; fineness modulus 4.1; water, 3.48 gal. per bag.

6. High alumina cement and silica aggregate, 1 to 2.67 by volume, plus a heat reducing admixture; fineness modulus 4.1; water, 3.28 gal. per bag.

7. Portland cement *B* and silica aggregate, 1 to 2.67 by volume; fineness modulus 4.1; water, 2.98 gal. per bag.

Samples 5 to 7 were 12 days old at the beginning of the test; moist cured for 2 days.

The fineness modulus for Sample 4 was reduced by adding 10 per cent (of weight of original sample) of material retained on and 2 per cent passing No. 100 sieve.

Aggregate

The aggregate used in these tests was a natural silica sand nominally of fineness modulus 4.0. A typical sieve analysis is as follows:

Retained on ⅜"	1 per cent
No. 4	18
8	43
16	62
39	82
50	97
100	99
Fineness Modulus	4.02

Results of Wear Tests

The graph shows the results of the wear test. The mix selected in 1933 (No. 2) is evidently superior to the modified one, (No. 1). The other samples were made with the richer mix, so they must be compared with No. 1 rather than No. 2. Sample No. 3 tested faily well in spite of the mistake in proportioning. (The last reading for No. 3 and No. 4 should be discarded, as the edge of the track was slightly broken down by a worn caster just before the end of the test.) A miniature concrete, proportioned for maximum density, might be expected to show better results in another test. The inference to be drawn regarding Sample No. 4 is not clear. Actually it was identical with No. 1, plus 12 per cent of material finer than a No. 50 sieve. The

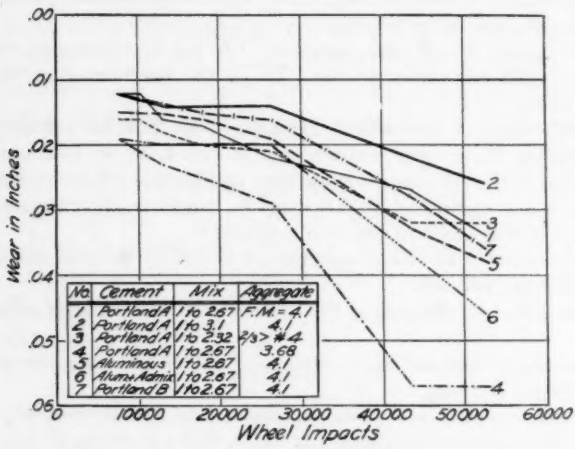

FIG. 1—WEAR OF SEVEN SAMPLE SEGMENTS OF TEST FLOOR

ACID TESTS

silica aggregate contains almost none of this size (the volume of voids being 37 per cent) so its effect should be to increase the density of the mix and also its workability. This section did puddle more quickly under the power float, but there was no apparent difference in troweling. It is possible that the fines in this mix were brought to the surface by the power float, and these might be worn down or knocked out more easily, but this could not be detected by an inspection of pieces cut from the floor. If the finish was actually denser, the indication is contrary to the assumption regarding Sample 3.

Samples 1, 5, and 7 are alike except that different kinds of cement were used. These samples gave results so nearly identical that it is safe to say the kind of cement used had no practical effect on the wearing quality of the floor.

Comparing Samples 5 and 6, the wearing quality of the alumina cement finish was slightly impaired by the admixture, but not to a prohibitive extent provided its resistance to acid action is not impaired.

After the completion of the wear test, three pieces were cut from each section of floor and subjected to acids of the greatest concentrations commercially available. The edges of the samples were painted with an acid-resistant paint so that when the acid was poured onto the surface, it stood from $\frac{1}{32}$ to $\frac{1}{16}$ in. deep.

H_2SO_4	—95%	Solution (36 normal)
HNO_3 69	—70%	Solution (16 normal)
HCl	35.5—37.5%	Solution (12 normal)

First one sample from each section was tested with all three acids. First HCl was poured on and allowed to stand for one hour; this was then washed clean and HNO_3 was applied in the same way; finally H_2SO_4. This was continued for 4 cycles, or a total of 12 hours. By this time all the samples showed sufficient reaction so that a good comparison could be made. This test was meant to duplicate in a general way what might happen to a floor when different acids are spilled on it and washed off. After the last application of acid, the samples were washed clean and allowed to dry.

The samples were inspected with a glass and a knife, and were rated on the basis of extent and depth of corrosion, and condition of the surface after the test.

Comparative Rating, with the best at the top:

Samples 2 and 7—Practically alike; the film of paste was gone from the surface, but the particles of aggregate were still embedded in the matrix. The surface of No. 7 was considerably softer than No. 2, indicating that in service the former would disintegrate more quickly. However, the surfaces of all these samples were sufficiently softened so that their wearing qualities would be materially impaired.

Samples 3 and 4—The surfaces were slightly more broken up and pitted. There was deeper corrosion on No. 4, but this sample was harder than No. 3, and even than No. 7, so it is but slightly inferior to them.

Sample 1—This sample was worn as much as No. 4 and was noticeably softer.

Samples 5 and 6—Very badly corroded; there were many loose particles. These samples were disintegrated to a depth of $\frac{1}{8}$ inch or more.

Since none of these samples was entirely satisfactory in withstanding this combination of acids, it was decided to determine whether any of them was particularly resistant to any one acid. In previous tests HAc (glacial acetic) had exhibited very little activity on concrete (although it was very destructive on samples which had an asphalt base) so this one was considered safe. On the other hand, HCl had proven to be so generally corrosive that it was thought useless to try it again on these samples. So the two remaining samples from each type of finish were tested with HNO_3 and H_2SO_4. The procedure was the same as in the first test, except that the acids were left standing for six two-hour periods, with intermediate washes, instead of twelve one-hour periods.

The samples tested with HNO_3 were rated as follows:

No. 3—The surface appeared to be virtually untouched, but it was soft enough to be scratched easily.

No. 1, 2, and 4—All of these had a few small pits in the surface but the maxtrix was not badly eaten away. The knife scratch penetrated about $\frac{1}{16}$ in.

No. 7, 5 and 6—The matrix was dissolved more deeply, so that pieces of aggregate were loose; No. 7 was noticeably better than 5 and 6.

The rating for the H_2SO_4 test is:

No. 3 showed very little attack.

No. 2 and 7—There was not much difference between them.

No. 1 and 4—The aggregate was a little more exposed on No. 1 and 4.

No. 5 and 6—The matrix was corroded much more in these two samples, but there was no disintegration.

The H_2SO_4 did not soften the surface as the other acids did. On all of the samples the surface was slightly chalky, but this was not deep, and the finish was still very hard underneath. This chalky surface was probably $CaSO_4$, which is practically insoluble and acts as a protective coating.

It has been observed that the silica sand did not deteriorate at all in any of these tests. It is an entirely satisfactory aggregate.

Some of the samples in the last two sets had wheel tracks on them, made by an inside wheel so the wear was not deep; only the film of paste was gone from the surface. In every instance this area was corroded more quickly and more deeply than the parts not worn by a wheel, even though there was a relatively smaller amount of paste exposed to the acid.

Differences in acid action seem to be only of degree. Their relative action is as follows: HCl is most severe, then HNO_3, H_2SO_4; and HAc is very slight.

There were not enough samples to make any determination of what strengths of acid solutions are most corrosive. It might be noted that in a previous test, made on only one kind of finish,

6N—H_2SO_4 was much worse than 36N

6N—HNO_3 was no worse than 16N

6N—HCl was very little worse than 12N

GENERAL SUMMARY

Acid Tests			Wear Test
3 Acids	HNO_3	H_2SO_4	
2, 7	3	3	2
3, 4	1, 2, 4	2, 7	1, 5, 7
1	7	1, 4	3
5, 6	5, 6	5, 6	6
			4

CONCLUSIONS

Results of the test made by the late Mr. Fairchild indicated that the most desirable fineness modulus of the aggregate was approximately 3.93, and the crushing strength of the concrete approximately

5500 p.s.i. The best mix seemed to be 1 part of cement to 3.1 parts of aggregate and 3½ gal. of water per bag of cement. This mix was arbitrarily changed to 1 to 2.67 to faciliate placing in isolated areas not reached by the power float.

The mix of portland cement and silica aggregate, 1 to 3.1, was so clearly the best in this latest wear test and stood up so well in all the acid tests, that its use has been adopted again in spite of the fact that it is not as easily worked. Sample 3, the miniature concrete, tested well enough to indicate that it should be carefully considered in another test. The alumina cement was so consistently inferior in the acid tests that it should not be used under those conditions. For ordinary wear it might well be used when a short curing period is required. All of these samples might react in an entirely different manner with weak solutions of acids.

In presenting this report with the foregoing data and discussion by Mr. Klock the committee has no illusions of finality. It hopes that those who have further data or suggestions will present them to the Institute. In presenting, for discussion only at this time, the practice outlined in the appendix the committee gives assurance that the recommendations are based not only on the tests reported by Fairchild in 1934 and by Mr. Klock at this time but on thousands of square feet of wearing surface placed in accordance with these recommendations. Such floors are giving perfect service under severe conditions of use and abuse.

In connection with the grading of aggregates, the committee finds it definitely established that it is desirable to omit particles passing the 50 mesh sieve, even though this will not necessarily give the densest concrete. The committee recommends further tests, especially with the more finely ground cements.

In general, the committee feels that the recommendations for wearing surfaces should: (1) be very brief; (2) give instructions for field labor that are simple to follow, in terms that are familiar to the average construction superintendent; (3) include specifications for material that are as rigid as is desirable.

The committee has been handicapped in trying to obtain aggregates in commercial quantities to meet its requirements. It is obvious that there is not sufficient market in the floor finish in most localities to justify dealers in stocking a separate aggregate for this purpose. With that in mind, limits were set on the fineness modulus, and it is hoped that the proportions of commercial aggregates may be varied to fit within the limits proposed.

Mr. Klock reports that the acid tests were "meant to duplicate in a general way what might happen to a floor when different acids were spilt on it and then washed off." The committee has reason to believe that for dilute acid solutions certain cements are better than others, but that pursuit of this subject would lead too far afield.

The committee records a word of caution against the use of power floats or steel trowels where sparks may produce a dangerous condition because of explosive gases.

Lastly: Two course work gives the contractor the opportunity to install wearing surface free from the vagaries of the weather, and disturbance by other trades. In the belief that an estimate of costs will result in a wider acceptance of such work, a schedule of labor cost in laying 1000 square feet of the recommended surface is given:

Preparing the slab (roughening and cleaning)			
32 man hours (common labor)	@	65c	$20.80
Laying bonding coat of mortar			
5 man hours (common labor)	@	65¢	3.25
Laying the finish itself (includes tamping, screeding, power floating and hand troweling)			
22 man hours (common labor)	@	65¢	14.30
12 man hours (skilled labor)	@	$1.25	15.00
Curing			
3 man hours (common labor)	@	65¢	1.95
Total labor cost per 1000 square feet			$55.30

APPENDIX—REPORT OF COMMITTEE 804

SUGGESTED RECOMMENDED PRACTICE FOR WEARING SURFACES FOR FLOORS

Clauses designated thus (A) refer to two course work only. Clauses designated thus (B) refer to monolithic finishes only. All other clauses refer to both types.

MATERIALS

Cement

Cement shall be of a standard brand meeting the requirements of the current A. S. T. M. specifications.

Aggregates

Aggregates shall conform to the Current A. S. T. M. "Tentative Specifications for Concrete Aggregates" except for grading and hardness.

Fine Aggregates* shall in addition meet the the following requirements:

*Where power floating is impractical combined aggregates with a F. M. as low as 3.5 will be permitted. In that case either the percentage of material passing the 30, 50, and the 100 sieve may be increased accordingly or the proportions may be varied to obtain that value.

Sieve Size	Percent Passing
¼"	100%
4	90—100%
8	70— 90%
16	60— 80%
30	40— 60%
50	5— 15%
100	0— 3%

Coarse Aggregates shall in addition meet the following requirements:

Sieve Size	Percent Passing
⅜"	100%
4	65—85%
8	40—60%
16	0—15%
30	0%

Proportions—1 part cement; 1⅓ parts sand; 2 parts of coarse aggregate.

Fineness Modulus (combined aggregate) 3.8 to 4.2 . All materials of each batch shall be measured carefully, preferably by weight.

Minimum Thickness—(A) The wearing surface shall have a minimum thickness of 1 in. (B) The wearing surface shall have a minimum thickness of ½ in.

Hardness—Aggregates shall be silica, traprock, granite, or other material of equivalent hardness.

Water

Water shall be clean and free from strong acid, alkali, or organic material. Total water, including water in aggregate, shall not exceed 4 gal. per bag of cement for power floating, and 5 gal. for hand floating.

Consistency

The mixture shall barely hold together and feel slightly damp to the touch.

Preparation of Base

(A) Thoroughly roughen surface of base to expose coarse aggregate. Remove all loose debris and fine particles with hose. (Air or water under 100 lb. pressure should be used where available.) Wet down exposed surface but in no case shall there be free water on the surface when the topping is being placed. Rub in thoroughly with a stiff broom (rattan) a thin layer of grout (1 cement to 1 clean mortar sand) of thick consistency, not more than 15 minutes prior to placing finish.

(B) The base should be thoroughly tamped. (The use of low slump concrete and vibrators is strongly recommended.) Screed base roughly, leaving the aggregate exposed to obtain a good bond with the finished surface. After the concrete has stiffened sufficiently (30 to 45 minutes) place the wearing surface material.

Placing Concrete

Mix each batch mechanically for 2 to 3 minutes, and screed to final position. Thoroughly roll, and float with wood, or power float to a compact and smooth surface. Let stand for not less than 30 to 45 minutes. Begin trowelling when the pressure from a finger ceases to make an indentation and trowel to final surface. The use of a power float, where available, is strongly recommended.

Curing

Keep wet for at least 7 days for standard portland cement and for 48 hours for high early strength cement, making certain that the entire surface is kept moist. Careful attention should be paid to the edges of the slab where masonry or other source of absorption or drying is present. Wet slab and cover with burlap and see that entire slab is wet continuously. Tough waterproof paper may be used if the joints are sealed, to prevent evaporation.

Curing should start as soon as the operation will not mar the finish, but not later than 3 hours after the final troweling, in order to eliminate spotty drying.

Discussion of the foregoing report will be welcome if received in triplicate by the Secretary of the Institute by December 1, 1938. For such discussion as may develop readers are referred to the JOURNAL *for February 1939.*

Vol. 35 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

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Discussion of a Report of Committee 804:

Concrete Wearing Surfaces for Floors*

BY R. T. GILES, E. W. SCRIPTURE, JR., F. E. RICHART, C. B. VANNIER, F. G. BALDWIN, AND CLOSURE BY THE COMMITTEE

BY R. T. GILES†

The report with reference to wearing surface leaves in doubt an important detail. Page 22 under "Preparation of Test Floor," the following curing procedure is described: "The test sections were cured under wet burlap, the standard portland cement mixtures for one week, and the high early for two days." If the burlap rested on the surface of the alumina cement sections this method is not in conformance with good practice and should never have been used. Such a curing method is contrary to information in all literature describing methods of curing for high alumina cements since it is desirable and necessary. if best results are desired, to employ a method which dissipates the heat of hydration. Covering with burlap acts as a blanket and retains the heat of hydration in the concrete. Good practice would involve the sprinkling of the surface of the concrete with water to resupply any mixing water lost by evaporation and sufficient additional water for evaporation to dissipate the heat of hydration. The effect of the curing method used on the wear and acid resistant properties of the concrete is difficult to evaluate. It very probably reduced the resistance to wear and acid.

Upon the first and a casual reading of the remarks on the resistance of the concretes to acid attack it appears that the conclusions are entirely too broad. Upon a more careful reading of the text no disagreement can be found with it, except to point out the danger of

*JOURNAL Amer. Concrete Inst., Sept. 1938; *Proceedings* Vol. 35, p. 21.
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drawing conclusions from one set of tests. Certain observations appear to be in order regarding these and other acid tests.

(1)—Neither portland nor high alumina cement would be satisfactory in acids of as high concentrations as used in these tests.

(2) Results obtained with an acid of high concentration do not indicate necessarily the results which may be expected with a lower concentration of the same acid.

(3) In general, concrete is seldom exposed to an acid having a normality higher than 1.0.

(4) The use of an acid of high concentration in an accelerated test may produce results that are misleading.

(5) The use of solutions made from pure chemicals in the laboratory may have no relation to conditions in a plant where impure chemicals are usually encountered.

The committee is to be congratulated on opening up for discussion this important use of concrete. It is possible the answer will necessitate consideration of concrete structures other than floors exposed to acid solution and that only results of actual installations in service will make it possible to draw any broad definite conclusions.

It would appear to be much safer to follow a method similar to the one used to determine the relative resistance of portland and high alumina cements to sulphuric and sulphurous acid solutions. A survey of a large number of concrete structures in coal mines was made. About 100 of these were made with high alumina cement. This survey was augmented by many inspections of concrete in other industries exposed to similar conditions and where the acid concentration was higher. These inspections include floors or other structures in rayon plants, powder plants, oil refineries, coke plants, copper smelters, gypsum mines and mills, industrial drains, paper mills, stacks and chinneys, ash pits, etc.

While the results of the survey and inspections disclosed the greater resistance of alumina cement there was a wide variation in the degree of resistance. This wide variation was also found in laboratory tests. Tests made on concrete specimens showed the alumina cement to be from 8 to 10 times as resistant as portland. On the other hand, linings in cast iron pipe exposed to a solution containing 5 per cent sulphuric acid and 5 per cent ferric sulphate showed a much greater difference. The lining made with portland cement was destroyed in five days while the lining made of high alumina cement was in good condition at the end of one year when the test was discontinued.

With this background it seems perfectly safe to draw a definite conclusion that alumina cement is more resistant to dilute solutions of sulphuric or sulphurous acids at atmospheric temperature than is portland cement. For elevated temperatures and stronger solutions the relative resistance of the two cements may be entirely different.

Before any conclusions are drawn, it is felt that a similar survey, augmented by inspections in various industries, should be made on concrete exposed to other acids.

BY E. W. SCRIPTURE, JR.*

Committee 804 has presented an interesting report of some tests carried out by Mr. Klock and has offered certain recommendations for wearing surfaces for floors.

The experimental investigations of Messrs. Fairchild and Klock are similar and confined to one or two factors involved in laying floors under specialized conditions. It would seem that they dealt with a particular type of service and that certain restrictions were placed on the materials and methods to be used in laying their floors which, while valuable contributions to the solution of the problem for the conditions with which they were concerned, need not be and prabably are not applicable to the vast majority of floors. The other basis for the recommendations is experience with "thousands of square feet of wearing surface" also presumably laid under the rather special restrictions imposed by the conditions of service. When it is considered that a building with one or two hundred thousand square feet of floor area is not unusual and that buildings are not unknown with more than a million feet of floor area, recommendations based on mere "thousands of square feet" seem to rest on a rather slender basis. It is hoped here that some contribution may be made to the subject of floors based on a record of continuous investigation of this subject in this laboratory over a period of some thirty years and field experience with hundreds of millions of square feet of floor area over a like period.

The criticism which is offered of this report of Committee 804 is three-fold. First, as noted above the recommendations rest on too narrow and restricted a basis. Second, certain of the data reported tend to be misleading because the conditions of test considered only very narrow and unusual conditions. Third, consideration has been given to a very limited number only of materials and methods: Many materials and methods which have been fairly generally accepted as good practice have been neglected.

*Master Builders Research Laboratories, Cleveland, Ohio.

Mr. Klock's tests, which are a continuation of Mr. Fairchild's investigations, may be said to deal with four variables: (1) Type of cement; (2) Mix; (3) Fineness modulus; and (4) water ratio. In view of the considerable number of different types of cement now available the data presented from tests of only two varieties certainly do not offer any reasonable basis for the selection of one particular type. Taken in conjunction with the work of Fairchild, these tests do seem to show the desirability of using a mix of approximately 1:3 provided that a suitably graded aggregate is available and that the floors are to be laid with a certain type of equipment, a power float, with labor which is skilled in this method of laying floors. It is doubtful whether either of these conditions could be fulfilled in the majority of cases. It may be pointed out that only one type of aggregate was used, a graded silica, which would not be available except in a relatively small area. There would also appear to be little doubt that an aggregate of fineness modulus about 4.0 is desirable if available. The recommendation that particles passing a 50-mesh sieve should be omitted does not appear to be well supported: the poor results secured in the one test made may be attributable more to the particular method used in laying the test panels than to any inherent defect of the mix. It seems curious, that in the recommendations based on these tests a maximum of 4 gal. per sack is set, whereas the tests show a maximum of 3.28 gals. per sack and this in the mix which is held up as an example of the bad effect of material through 50 mesh.

It should be remembered that high alumina cement has certain peculiarities and behaves quite differently from either normal portland cement or high early strength cement. The most striking peculiarity is a very rapid evolution of heat at a particularly early period. Unless measures are taken to control this heat the cement is likely to lose much of its strength and surface may become powdery. The manufacturers' specifications for high alumina cement call for sprinkling with water during the period of high heat evolution and specifically caution against covering the floor. Again the setting characteristics of high alumina cement differ from those of portland cements. Yet in the tests reported the high alumina and the portland cement panels were treated identically except that the high alumina cement was cured for two days only. No provision was made for sprinkling the high alumina panel and the test area was covered; directly contrary to the manufacturer's specification.* With respect to the shorter time of curing, this should make no difference with the reg-

*"Do not cure by ponding the surface or by covering with wet sawdust, wet burlap or other coverings."

ular high alumina cement since it should have developed substantially all of its strength within 24 hours, but with the modified high alumina, designed to control heat evolution and delay the development of strength, this factor may have had some influence.

It would seem quite possible that the failure to secure good resistance to abrasion with the high alumina cements is directly attributable to improper treatment of these cements. Furthermore, the superiority shown by the portland cement mixes is directly contrary to data secured in this and other laboratories and to experience in the field, where these special cements were handled in an appropriate manner. Some test results from this laboratory on the high alumina cement and the modified high alumina cement with respect to compressive strength and abrasion may be cited as examples:

TABLE 1

1:3 Mortar	Water c.c. for 1100 g. Cement	Compressive Strength—Lbs. per Sq. In.			
		1 Day	3 Days	7 Days	28 Days
Portland Cement	500	....	1780	2515	4610
High Alumina Cement	500	....	7475	6650	6900
Modified High Alumina Cement	440	....	2990	5410	6740
Portland Cement	500	525	1380	3100	4550
High Alumina Cement	505	5725	6140	6235	6790
Modified High Alumina Cement	460	3436	4484	4770	5980

	Depth of Abrasion—Inches Pressure 100 lbs.—15 min.
1:3 Silica Aggregate Mortar	
Portland cement	0.009
High Alumina Cement	0.007
Modified High Alumina Cement	0.007
1:3 Mortar + 40 lb./100 sq. ft. Metallic Aggregate	
Portland cement	0.008
High Alumina Cement	0.004
Modified High Alumina Cement	0.004

The data given in the committee report would lead to the conclusion that high alumina cements were less resistant to corrosion than portland cements. This again is directly contrary to the findings of most laboratories and to the evidence of many years of field experience. Some test data from this laboratory on both the regular and the modified high alumina cements under corrosive conditions confirm the field evidence.

TABLE 2—5 MONTHS CORROSION (FOLLOWING 28 DAYS CURING)

In Solution	*Untreated* Compressive Strength—Lbs. per Sq. In.		
	Portland Cement	High Alumina Cement	Modified High Alumina Cement
8% Magnesium Sulphate	1820	6520	4550
1% Nitric Acid	0	3690	3720
	Treated with Corrosion Resistant Composition		
8% Magnesium Sulphate	4295	5885	5725
1% Nitric Acid	1558	3530	3500
1% Acetic Acid	1877	4260	5630
1% Hydrochloric Acid	3022	4580	4360
1% Sulphuric Acid	1275	2860	3500

An explanation of the anomalous results given in the committee report may be that all the corrosion tests were made with acids ranging from 6 normal to 36 normal, that is, concentrated acids. Both the portland and the high alumina cements will react readily with acids of this concentration so that it is not surprising that they should fail under these conditions. The relatively inferior performance of the high alumina cement mixes may again be attributable to improper treatment of this cement in making the test specimens. While tests with concentrated acids may simulate the spillage of chemicals on the floor it does not simulate the conditions met in most of the industries where an acid-resistant floor is desired.

No consideration has been given to many materials available for use in floors. It would seem that other types of cement should be considered, particularly the high early strength cements (of the portland type) which offer advantages with respect to shortening the time of curing. While the subject of admixtures may be over controversial for a recommended practice report it must be admitted that some of these may be desirable. Calcium chloride as an aid to curing has been recognized at least to the point of adoption of A. S. T. M. standards and there are others which would seem to have some place in floor practice.

Only one type of aggregate, namely a crushed and graded silica, has been studied. This laboratory is particularly familiar with metallic aggregates and some test data may be cited as an example of the results obtainable with aggregates other than silica. It may be mentioned that millions of square feet of floor with excellent performance records have been laid in all parts of the country with metallic aggregates.

A comparison of a 1:1½:1½ silica aggregate mix with and without two proportions of metallic aggregate on the surface is as follows:

Test Number	Depth of Abrasion—Inches		
	No Metallic Aggregate	Metallic Aggregate	
		60 Lb./100 Sq. Ft.	120 Lb./100 Sq. Ft.
1	.019	.011	.007
2	.016	.011	.007
Average	.017	.011	.007

(Slabs cured 10 days, dried 7 days, Abrasion 15 min. at 100 lbs.)

The committee has studied one method of laying a floor, using a very dry mix and a mechanical float. The mechanical float is by no means the only device which can be used to aid in laying a good floor. Various devices for tamping, vibrators, rollers, and other methods of

laying low water-cement ratio floors have been used with success. Perhaps the mechanical float is the most satisfactory means of accomplishing the desired result but the committee has certainly adduced no evidence to that effect.

As an example of other methods of laying floors reference is made to an article* from this laboratory describing the use of a special type of tamp (grill). Subsequently an extensive investigation of the adaptation of the principle of this type of tamp to a vibrator was carried out. Some unpublished data showing a comparison between this method and the mechanical float are as follows:

TABLE 3—COMPARISON OF FINISHING METHODS

Mix: 1 part portland cement: 3 parts silica aggregate.
Curing: 1 week in moist closet: 1 week in air.
Abrasion: 15 min. 100 lb. pressure.

Shake	or Vibrated	Mechanical or Hand Floating	Depth of Abrasion Inches
None	Vibrated	Mechanical	.007
None	Vibrated	Hand	.009
None	Tamped*	Mechanical	.007
None	Tamped	Hand	.009
120 Lb. per 100 sq. ft.	Vibrated**	Mechanical	.003
100 Lb. per 100 sq. ft.	Vibrated	Hand	.004

*Tamped—grill tamp used as described in earlier article.
**Vibrated—vibrating platform with grill surface.

These figures illustrated the advantage of mechanical floating compared with hand floating but they also show that a metallic aggregate shake produces greater differences in resistance to abrasion than does the method of finishing.

The committee appears to have a preference for two course construction. While there is much to be said for it there is something to be said on the other side. Just what the vagaries of the weather have to do with the question is not clear. Monolithic work, (forming of the concrete floor as a single unit without a separate wearing course mix has not even been considered by the committee) especially if finished with a suitable shake, has much to recommend it.

Some comment may be made respecting the details of the suggested recommended practice. The paragraph on cement does not give any information regarding the type of cement to be used: as far as this recommendation is concerned a masonry cement might be employed. The aggregate requirement is so rigid that it would probably be impossible to comply with it in many localities. The provision for 5-15 per cent through 50 mesh is not consistent with the earlier recommendation of the committee that material through 50 mesh be omitted. The

*For test method see "Metallic Aggregates in Concrete Floors" by Edward W. Scripture, JOURNAL Amer. Concrete Inst., Sept.-Oct. 1936; *Proceedings* Vol. 33, p. 17.

proportions of the mix are too rigid. There does not seem to be any reason why the mix to be used for floor topping should not be specified in much the same manner as has been used for concrete, namely, by fixing the water cement ratio and adding aggregate to produce the consistency required for hand or mechanical finishing. For many purposes it is questionable whether the hardness of the aggregate materially affects the serviceability of the floor: there is much to be said for a soft malleable aggregate such as iron in comparison with a hard brittle aggregate. The requirements of the report would preclude the use of light-weight aggregates although these have been used successfully in floors and pavements.

As previously mentioned the water requirements are not consistent with the experimental data. The consistency of the mix as specified is satisfactory for mechanical floating but would not be so for hand floating.

Under placing, the recommendations appear to call for rolling with some type of roller not stated. Whereas it may be desirable to use a roller, satisfactory floors can be laid without rolling by the use of tamps. Tamping, and presumably rolling, should precede screeding not follow it. If a dry mix as recommended is used the first trowelling can follow immediately after mechanical floating and there is no need to wait 30 to 45 minutes. On the other hand, there should be a wait between the first troweling and final trowelling until the floor has attained just the proper hardness.

It is also stated that curing should start not later than 3 hours after final trowelling. On most jobs this is hardly practical where the final trowelling is completed late in the afternoon. It is usual practice to start applying curing measures on the following morning.

It seems that the report of the committee should be based on much wider experience under more varied conditions, and that it should take cognizance of methods and materials which have not been considered. While the desirability of brevity and simplicity are self evident, that of rigidity is not. The suggested recommended practice does not appear to be clear, sufficiently comprehensive or sufficiently flexible for general use.

BY F. E. RICHART*

In reading the description of floor tests given in this report, it occurs to me that information on a local dairy floor might interest JOURNAL readers. This floor is in the cold room of the Urbana Pure Milk Co., and has been in service nearly six years. The concrete was

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made with Lumnite cement, of approximately 1:2:3 mix. The floor was given a float finish and water was applied to the surface at frequent intervals during the first 12 hours after placing.

The floor at present is in excellent condition. It shows some wear due to the small iron wheels or casters used on the bottle trucks, but no disintegration otherwise. The floor is generally wet with water and milk. The room is kept at about 45° F.

Considering the usual severe attack of lactic acid on concrete in dairies, the performance of this floor is outstandingly good.

BY C. B. VANNIER*

The information contained in the report is necessary to the construction of first-class floors, and should be of great value to anyone who is interested in building better concrete floors. On such information (with the exception of the results of the acid tests) I based my work when I began floor construction as a specialty. However I doubt if the report supplies a sufficient basis for floor construction in various localities with different types and classes of aggregates. The recommended grading for aggregates, does not conform to the sieve analysis of the aggregates giving me the best results. Percentages retained on each sieve show the higher percentages in the middle, tapering both ways, while I have found that my best gradings show up more like a wedge, heavy at the top, and tapering toward the bottom. This not only tends to greater reduction in voids, but also reduces the surface area.

My approach to the problem of securing uniform results was not that of the laboratory, but of the man in the field, who has his specifications furnished, and has to use the materials available, keep the cost as low as possible, and still produce a first class job. I finally assembled about the same information contained in the report of Committee 804, but found that it did not supply me with the answer to two problems encountered in the field: (1) What proportions to use of sand and coarse aggregate where the gradings did not fall within the limits recommended; (2) How much cement to use with a fixed water content, and a given aggregate.

I got together many samples of aggregates, and made a sieve analysis of each. Then I made various combinations, keeping careful record of the percentages of each, until I had reached a result nearest to the best recommended grading, all proportions by weight.

I am now using a gravel combination from a pit about fifty miles from Buffalo, that has a fineness modulus of 4.4, an absolute volume

*Floor Contractor, Buffalo, N Y

of .71 (loose dry measurement) with less than 5 per cent passing a 48 sieve. This gives good cement economy, and shows fine density. I am using this material under metalicron shakes, as well as for plain finish.

Some years before going into floor work I had worked out a field method of determining the voids in aggregates that proved accurate to within less than 2 per cent after being checked by several ⅓ yard cubes. I use this method in determining the absolute volume of the aggregates for all my floor mixes. It is similar to the method described in the *National Sand and Gravel Association*, Bulletin No. 4, Aug., 1928, except that instead of pouring water into the aggregates, I pour the aggregates into the water, and calculate the volumn of the water displaced. All my formulas are based on loose dry measurements.

I have found that if the absolute volume of the water and cement are equal to the voids in the loose dry aggregates, that the mix is just right for power floating.

A decrease of 5 per cent will make the mix too harsh to trowel properly, and an increase of 5 per cent brings too much mortar to the surface.

By these methods, I am not only able to get what I believe is the best floor out of a given aggregate, but also, I am able to calculate my quantities accurately—of value in estimating.

In the fall of 1936, I used the first model of the Kelley power float with the tamping attachment, on about 96,000 sq. ft. of 5 in. plain 1:2:4 concrete floor slab. Up to that time, the power float did not seem very practical for floating monolithic slabs, but this experimental machine did a fine job, and the floor is in excellent condition today after continuous trucking service. Since then, I have installed a number of monolithic jobs using a shake of Metalicron, and have obtained excellent results by floating with the new tamper type of power float.

I have just completed a job of 124,000 sq. ft. where I power floated into the surface a shake of well graded coarse grit and sand having a fineness modulus of 3.5. This floor has a fine finish and I have every reason to believe will give excellent service for a floor in this low price class. Pieces cut from the floor show excellent densification. The concrete used was 1:2:4 mix, and the aggregates were graded from one inch down, the gravel being 50 per cent No. 1 and 50 per cent No. 2, which gave a low void content with the materials used.

I hope that discussion of this report will lead to further research on grading of aggregates and designing of floor mixes, by persons better qualified than myself to do this type of investigation.

BY F. G. BALDWIN*

It seems in the matter of mixes of aggregate that the common practice of using sands of a given fineness modulus for hard wearing floor surfaces is not sufficiently accurate. Sands are so often stripped for sand blasting or molding or whatnot that the resultant product is apt to suffer. My work has consisted in determining the actual quantity of each size of particle required to form the densest grading that would allow for the subsequent addition of the water-cement paste. This necessitated definite quantities. I see no other way to accomplish this accuracy than to combine the several sizes by artificial means whether it is done in a central plant or done in the field. I actually tested samples having identical fineness modulae but differing in the internal structure and had very noticeable variation in compressive strength.

I contend that the accuracy of the mix for concrete hard wearing surfaces must be of precise uniformity lest a batch of the minimum come immediately adjacent to one of the maximum hardness, for, as wear occurs the softer spot is subjected to the maximum of impact. I do not see how a specification can be enforced unless the product is divided and recombined with accurate control.

CLOSURE DISCUSSION BY THE COMMITTEE†

The committee expresses its thanks to those who have offered suggestions, and would like to offer further suggestions with the hope that some graduate student looking for a subject for research would tackle one of these problems:

(1) What measure of hardness can be applied to aggregates that would be proportional to the wear of the aggregate in the finished floor? We don't know.

(2) What percentage of voids in the aggregate would make the best floor? The committee feels that less than 30 per cent would produce better results than most of the aggregates we used, where the percentage of voids in the combined aggregate in the dry rodded state varied from that figure to about 40 per cent.

(3) What percentage of the voids in the aggregate should be filled with paste?

(4) What strength concrete would make the best floor? The committee, based upon its own tests and tests reported to us by others, feels that a value around 5,000 lb. based on W/C curve is the best to be obtained. We are wondering whether that 5,000 lb. limit is due to the fact that the equipment for placing stronger concrete had not been perfected at the time the tests were made, so that the stronger concrete was not properly rolled, tamped, and trowelled.

At the first meeting of the committee, with the flush of amateurish ignorance, we decided to test every brand of cement, every patented

*The Maximent Co., Cincinnati, Ohio.
†J. Fruchtbaum, Chairman.

cure-all, and every theory and idea that in any way might be connected with the use of concrete in general and floor finishes in particular.

At about the third or fourth meeting, when we began to face realities, we limited ourselves to an unpatented, economical, and durable concrete wearing surface. We decided, however, that we would look over and make occasional tests on other types. The various products and types we examined, although they were more expensive, were not, in our opinion, the equal of the floor we were producing in accordance with our standards.

For example, we found that while the addition of a metallic aggregate reduced the wear where poor aggregate was used, the benefit received with hard aggregate was hardly discernible. Certainly a fraction of its cost would more than pay the difference between good and poor aggregate.

The tests on the wearing surfaces covered the following aggregates:

(1) Iron ore tailings
(2) Crushed quartz
(3) Silica sand
(4) Trap rock and silica sand
(5) Local sand and trap rock
(6) St. Lawrence River sand and trap rock

The grading was varied from a F. M. of 3.12 to 4.95. It was the unanimous feeling of the committee (seconded by many gravel producers) that this selection was sufficiently broad to be representative of material generally available.

With reference to the acid resistance of floors, we can only refer to the following from the report: "Mr. Klock reports that the acid tests were 'meant to duplicate in a general way what might happen to a floor when different acids were spilled on it and then washed off.' The committee has reason to believe that for dilute acid solutions certain cements are better than other, but that pursuit of this subject would lead too far afield."

Quoting again from the report "The kind of cement used had no practical effect on the wearing quality of the floor." This bears out the thought that the cement paste serves only as a binder, and has no bearing on the wearing resistance of the floor except insofar as it determines the binding quality of the paste.

The discussion on the curing of the Lumnite cement is for emphasis only, as the manufacturer knew that the curing was done in accordance with his specifications, under the instruction of their Mr. McDonald. We regret not emphasizing the point in our report, but we were aiming

at brevity. There can be no possible advantage however in adding an expensive retarding admixture to high alumina cement in order to obtain results that are not the equal of the more economical methods suggested in the report.

Incidentally the committee has no solution for a concrete wearing surface that will be acid resisting. It would recommend that a different type of finish be used where acid resistance is essential, and that where concrete must be used, the mix should be as lean as possible and the aggregate should be acid resistant.

Dr. Scripture states: "These tests do seem to show the desirability of using a mix of approximately 1:3 provided that a suitably graded aggregate is available and that the floors are to be laid with a certain type of equipment, a power float, with labor which is skilled in this method of laying floors."

Again, from our report "In general, the committee feels that the recommendations for wearing surfaces should: (1) be very brief, (2) give instructions for field labor that are simple to follow, in terms that are familiar to the average construction superintendent; (3) include specifications for material that are as rigid as is desirable."

To put it more strongly, the committee suggests a practice for producing a good floor that is very economical. Materials are such a small fraction of the cost of patented products which the committee recommendations would obviate. The rigid specifications that it places on the material are such as to increase the cost a very small fraction of the cost of patented products offered to remedy bad conditions which the committee would do away with.

A power float is recommended because, in the opinion of the committee it is the best equipment capable of laying a first-class floor at a reasonably low cost. Our experience has shown us that to roll the topping with an ordinary lawn roller compacts it very well. A special grill tamp might do it as well or better; or vacuum mats might be used to remove excess water and compact the material. However, all of these things are only preparatory to the use of the float. The committee feels that with the vacuum method of removing excess water, as with the burlap method, there is a possibility of drawing too much fine material to the surface, thereby increasing the likelihood of dusting and crazing. However, where control over mixing the finish is difficult, some method of removing excess water would be of great advantage.

To Dr. Scripture's point that "many materials and methods which have been fairly generally accepted as good practice have been

neglected, we can only reiterate that we are confident a real improvement in the quality of floor topping can be made over the fairly generally accepted good practice if this method is followed. Our recommendation is strict because we think it needs to be. A suggestion to pick a water-cement ratio and then add as much aggregate as will produce a desired consistency would be hardly a recommendation at all. It is true that the proportions of 1:1⅓:2 will yield an unworkable mix if the grading of the aggregate is markedly different from the fineness modulus of 4.0 which we recommend. For instance, if the particles of coarse aggregate are largely one size so that the voids are greater, a mix of 1:1⅓:1½ or 1:1½:1½ would be better; if the fine aggregate has 20 per cent or 25 per cent passing a No. 50 sieve, 1:1:1½ or 1:1:2 would be better. The point is, neither of these floors would be as good as the one we have recommended according to all the data we have gathered up to now. If the specification for grading can't be met, then the proportions should be changed, and properly, by trial and error. But it should be recognized that such a change will be made at a sacrifice of potential quality.

The term, monolithic floor, is defined by Dr. Scripture as a trowelled concrete slab with no added topping material. If we adhere strictly to that definition the best we can hope for is a 3,000 lb. concrete wearing surface (in most cases only a 2,500 lb. or 2,000 lb concrete). Regardless of name or definition, this is only a very mediocre surface, suitable only where little or no traffic is to be expected. The committee has ruled that particular type of finish out altogether. The two course construction in which the topping is placed before the slab has set (called monolithic by the committee) has the following disadvantages, in addition to those given in the report:

(1) Excess water in the slab is sure to be drawn up by the dry topping and bring fine material with it to the surface during the floating operation.

(2) If the topping is included in the designed thickness of the slab, the surface cannot be repaired or replaced without weakening the floor.

These will explain the preference of the committee for bonded finish.

In conclusion let us say again that we have attempted to describe and recommend the floor which we feel gives maximum service at a minimum cost. Our recommendations give our viewpoint. It may be of interest, however, that Mr. Loring, working independently at Cincinnati, arrived at conclusions almost identical with the rest of the committee. Added to that are the reports of industrial engineers who have been working in their particular plants on the same problem.

This floor is our first choice as a concrete wearing surface. It may be that other recommendations will be desirable and feasible for those

situations where the requirements of traffic are less severe, or where greater economy is demanded.

One more statement should be added in fairness to Mr. Knight, a member of the committee, who happens to be interested in the sale of equipment recommended in the report. Mr. Knight volunteered to serve on the committee under pressure and has taken no active part in writing the recommendations. His advice and observations were valuable to the committee.

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Tests of Reinforced Concrete Columns Under Sustained Loading*

By F. E. Richart and R. H. Heitman†

MEMBERS AMERICAN CONCRETE INSTITUTE

SYNOPSIS

Tests on reinforced concrete columns held under sustained designed loads for six years show little increase in steel stresses due to plastic flow of concrete after the first year of loading. These columns, tested to failure after six years of design loading, showed the same strength as companion columns that had not been under load. The strength of the six year old columns followed the same general strength relation found in the original A. C. I. column tests reported in 1931-32. The effectiveness of the spiral from these last tests was about 2.2.

Plain columns, stored six years in air, then placed under a sustained load of ¼ the cylinder strength, showed very little plastic flow in one year, as compared with similar columns loaded after 56 days of moist curing.

The foregoing results verify previous statements that the principal effect of plastic flow occurs during the first year of loading and that old, well-dried concrete will develop very little flow. They also extend the general results of the A. C. I. tests to six-year-old columns.

In the investigation of reinforced concrete columns sponsored by the American Concrete Institute in 1930, one group of tests (Series 3) was made to study the effect of shrinkage and plastic flow or creep upon the stresses in the two materials and upon the strength of the columns at failure.

The tests were made at the University of Illinois and Lehigh University; those at Illinois included 153 columns, 8¼-in. in diameter

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‡See Second and Fourth Progress Reports on Column Tests at the University of Illinois, American Concrete Institute Journal, March, 1931; *Proceedings* V. 27, p. 761 and January, 1932, *Proceedings* V. 28, p. 279. Also Bulletin 267, University of Illinois, Eng. Expt. Sta.

and 5 ft. long. Of these, 60 reinforced columns were held for a year under design loads, 48 were duplicate columns held in similar storage but under no load, and the remaining 45 were of plain concrete. All but 26 of the reinforced columns and 6 of the plain columns were tested to failure at the end of the year. The remainder have been held under observation for six years or more and form the subject of this report.

SIX-YEAR TESTS ON REINFORCED COLUMNS

As noted in previous reports‡ these columns were made with three grades of concrete having nominal strengths of 2000, 3500, and 5000 p.s.i. They were cured for 56 days in a moist room, then stored in the large crane-bay of the laboratory and subjected to sustained load and shrinkage observations. The sustained loads were applied by means of a group of car springs and tie rods arranged so that two companion columns could be subjected to a sustained load with each set of springs. The springs were calibrated and load was applied to the columns by tighting the nuts on the four tie rods. The car springs were checked at intervals to make sure that the calibration did not change with a long period of loading.

A complete set of strain measurements was taken on the columns immediately before and after applying the load, then generally at periods of 1, 3, 7, 14 and 28 days, and thereafter at intervals of several months. Measurements were taken on a total of sixteen gage lengths on a column. The working loads were calculated from the design formulas of the 1928 American Concrete Institute Building Code and (for four of the columns noted) of the 1916 New York City Building Code.

The strain measurements on the columns, which have previously been reported for the period of one year, were continued on the group of 26 reinforced columns for about six years and the significant results of these observations are listed in Table 1. The steel stresses after loading periods of 1, 2, 3, 4, 5 and 6 years have been tabulated together with recovery reading three days after the removal of the sustained load. The variations in the steel stresses in the unloaded companion columns over the 6-year period were negligible.

Since the columns showed very little change of strain in the later years except that due to seasonal variations in humidity and temperature, the tests were discontinued and the columns were released and then loaded to failure in the testing machine. It is of interest that very few cracks were noted when the columns were released from the loading rigs although the shape of the stress-strain curves for these columns indicated that such cracks existed. When the columns were

TABLE 1—STRESSES DUE TO 6-YEAR SUSTAINED LOADS ON COLUMNS

All columns 8¼ in. in diameter, 8 in. core diameter, 5 ft. long. Columns moist cured 56 days, then held under sustained load in air of laboratory for 6 years. Load then released and columns tested to failure. Sustained loads are design loads by 1928 A. C. I. code and (for those marked*) 1916 N. Y. City Code. All reinforcement of intermediate grade steel.

Col. No. indicates { concrete strength: A = 2000, B = 3500, C = 5000 p.s.i.; vertical steel, 1, 4 or 6 per cent; spiral steel, 1 or 2 per cent

Column No.	Sustained Load Lb.	Steel Stress, f_s, in Loaded Columns—p.s.i.							
		When First Loaded	After 1 Yr.	After 2 Yrs.	After 3 Yrs.	After 4 Yrs.	After 5 Yrs.	After 6 Yrs.	Load Released
A 41	64000	9400	19200	20100	20200	20500	18300	19200	2200
B 11	52400	9800	25400	26500	26500	27900	.....	27400	4200
B 12	52400	7000	26700	27300	26900	28600	29500	28800	8300
B 12*	67000	11100	30800	32000	32400	33800	35400	32700	7700
B 42	79000	9800	22200	23200	22900	23600	22700	23200	1000
B 62	108600	13100	23500	24300	23800	24800	.....	22700	0
B 62*	75200	8300	15000	15900	15400	16500	.....	16700	300
C 41	96000	10600	25000	26500	26100	27100	28900	27400	1400

TABLE 2—STRENGTH OF 6-YEAR OLD REINFORCED CONCRETE COLUMNS

As indicated in Table 1, sixteen of these columns had been under design load for six years, and ten had stood unloaded the same period. Average yield point of vertical steel, 1% reinforcement, 45600 p.s.i.; 4%, 53400 p.s.i.; 6%, 45200 p.s.i. Spiral steel, 1%, 49400 p.s.i.; 2%, 48200 p.s.i. Values given are average for two companion columns. C = 0.85.

Nominal Design			After 6-Yr. Loading					After 6 Yrs.—Not Loaded				
f'_c p.s.i.	% Reinf. Vert.	% Reinf. Spiral	Total P/A p.s.i.	Vertical pf_s p.s.i.	Conc. Cf'_c $(1-p)$ p.s.i.	Spiral $kp'f'_s$ p.s.i.	Coef. k	Total P/A p.s.i.	Vertical pf_s p.s.i.	Conc. Cf'_c $(1-p)$ p.s.i.	Spiral $kp'f'_s$ p.s.i.	Coef. k
2000	4	1	5600	2120	2085	1395	2.28	5690	2120	1990	1580	2.58
3500	1	1	5430	715	3060	1660	2.71	....	....	....	....	....
3500	1	2	6240	715	3480	2045	2.12	6070	715	3435	1930	2.00
3500*	1	2	6550	715	3480	2355	2.44	....	....	....	....	....
3500	4	1	6480	2120	3305	1055	1.72	6650	2120	3265	1265	2.07
3500	6	1	7975	2690	3280	2005	2.04	8425	2690	3190	2545	2.64
3500*	6	1	8060	2690	3110	2260	2.34	....	....	....	....	....
5000	4	1	7985	2120	4385	1480	2.42	7640	2120	4400	1120	1.83
Average value of k							2.26					2.22

Ratio $\frac{P/A \text{ for unloaded columns}}{P/A \text{ for loaded columns}}$

2000	4	1	1.016
3500	1	2	0.951
3500	4	1	1.026
3500	6	1	1.051
5000	4	1	0.958
Average			1.000

*Indicates sustained loading under 1916 N. Y. City Code.

again subjected to load in the testing machine it is evident that the cracks closed up. This was shown by the decreasing rate of deformation as the load was increased up to the design load. Beyond the design load the shape of the stress-strain curve was similar to that for companion columns that had stood unloaded during the 6-year period. The columns were loaded to failure with about ten increments

TABLE 3—VALUES OF "SUSTAINED MODULUS OF ELASTICITY," E'_c OF CONCRETE IN COLUMNS UNDER 6-YEAR SUSTAINED LOADING

Col. No.	Nominal Conc. Str. p.s.i.	Steel Area in.²	Conc. Area in.²	Design Load lb.	Steel Stress lb./in.²	Load on Steel,† lb.	Load on Conc. lb.	Conc. Stress† lb./in.²	Conc. Str. Millionths	E'_c Millions p.s.i.	$n' = \frac{E_s}{E'_c}$	Ratio n'/n
A 41	2000	2.00	48.27	64,000	19,200	38,400	25,600	530	695	.763	39.4	3.7
B 11	3500	0.78	49.49	52,400	27,400	21,400	35,000	707	898	.788	38.1	
B 12	3500	0.78	49.49	52,400	28,800	22,400	30,000	606	895	.678	43.6	
B 12*	3500	0.78	49.49	67,000	32,700	25,500	41,500	838	1054	.794	37.8	
B 41	3500	2.00	48.27	79,000	23,200	46,400	32,600	676	829	.815	36.8	
B 62	3500	3.00	47.27	108,600	22,700	68,100	40,500	857	810	.946	31.7	
B 62*	3500	3.00	47.27	75,200	16,700	50,100	25,100	532	595	.893	33.6	
									Ave.		36.9	4.6
C 41	5000	2.00	48.27	96,000	27,400	54,800	41,200	832	1174	.708	42.3	5.5

*N. Y. C. Loading. †At end of 6 Yr. Loading.

TABLE 4—AVERAGE STRENGTH AND MODULUS OF ELASTICITY OF CONTROL CYLINDERS

Design Str. p.s.i.	Cylinder Strength p.s.i.			Mod. of Elast., Millions p.s.i.			Ratio			
	56 Day f'_{c1}	1 Yr., 56 Day f'_{c2}	6 Yr. f'_{c3}	56 Day E_1	1 Yr., 56 Day E_2	6 Yr. E_3	$\frac{f'_{c2}}{f'_{c1}}$	$\frac{f'_{c3}}{f'_{c1}}$	$\frac{E_2}{E_1}$	$\frac{E_3}{E_1}$
2000	2200	2665	2500	2.830	2.985	3.260	1.21	1.14	1.05	1.15
3500	3730	4510	4075	3.800	3.980	4.170	1.21	1.09	1.05	1.10
5000	5460	6135	5380	3.290	4.170	4.630	1.12	0.98	0.97	1.08
Ave.							1.18	1.07	1.03	1.11

of loading and each column was tested to failure in a period of three or four hours.

Table 2 gives the principal results of the tests to failure. A comparison of the ultimate strengths of the columns which have been loaded and those which have stood unloaded for the 6-year period shows practically no difference in the two cases. An analysis of the division of the load between the three load carrying elements, the concrete, the vertical steel, and the spiral steel is also given. The average value of the effectiveness factor for the spiral (denoted by k) was found to be 2.26 for the loaded columns and 2.22 for those that had stood unloaded.

Table 3 shows average values of the modulus of elasticity and strength of control cylinders which were stored with the columns and tested when six years old; also similar values for cylinders made with other columns of the series which were tested at ages of 56 days and 14 months. The table also shows values of the "sustained modulus" of elasticity of concrete in the columns after six years under load. This modulus is computed by subtracting the steel stress computed from measured strains from the total applied load to get the portion carried by the concrete. This unit load on the concrete was then

divided by the average concrete strain to determine the sustained modulus of elasticity of the concrete. This gives a value of n' varying from about 37 for the 3500-lb. concrete to 42 for the 5000-lb. concrete. The ratio of the "sustained" value of n to the initial value ranges from 3.7 to 5.5 for the three grades of concrete used.

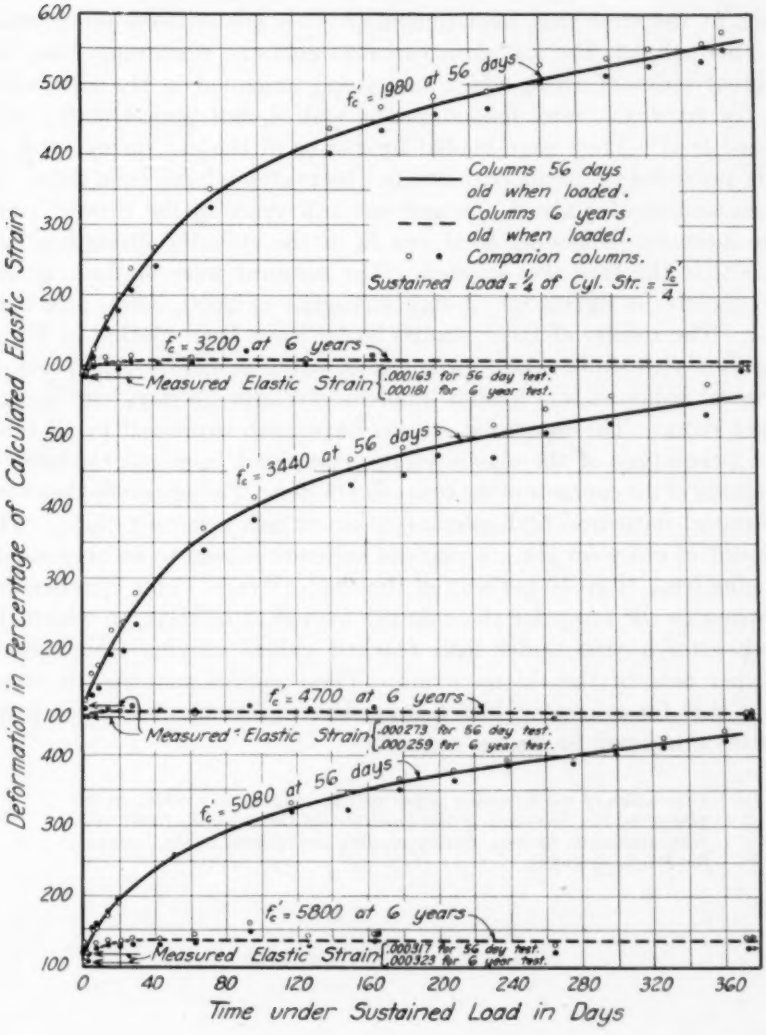
Deformation in Percentage of Calculated Elastic Strain
Time under Sustained Load in Days
f'c = 1980 at 56 days
f'c = 3200 at 6 years
Measured Elastic Strain .000163 for 56 day test. .000181 for 6 year test.
f'c = 3440 at 56 days
f'c = 4700 at 6 years
Measured Elastic Strain .000273 for 56 day test. .000259 for 6 year test.
f'c = 5080 at 56 days
f'c = 5800 at 6 years
Measured Elastic Strain .000317 for 56 day test. .000329 for 6 year test.
Columns 56 days old when loaded.
Columns 6 years old when loaded.
Companion columns.
Sustained Load = 1/4 of Cyl. Str. = f'c/4

FIG. 1

Table 4 gives values of strength and modulus of elasticity of concrete cylinders (1) tested moist after 56 days moist curing, (2) after 56 days moist curing and 1 year in air and (3) after 56 days moist curing and six years in air.

CREEP TESTS ON PLAIN CONCRETE COLUMNS

Various experimenters have stated that the amount of creep determined in plain concrete decreases with increase in age of the test piece at the time that load is applied. To secure some information on this subject, the six plain concrete columns remaining from the original series of 1930, which had stood unloaded in the laboratory air for six years, were loaded at the end of that period with a sustained load. They were loaded by means of the car springs and tie rods previously mentioned. Strain observations have been taken on these columns for about one and one-half years to the present time. The intensity of loading used was ¼ of the cylinder strength at the time that the load was applied. The columns were of three grades of concrete, designed for 56-day strengths of 2000, 3500, and 5000 p.s.i. The results of these studies so far have been plotted in Fig. 1 together with similar data for columns tested in 1931. This previous series of columns was loaded immediately after 56 days curing in a moist room. The measured strains have been expressed in all cases as a percentage of the elastic strain, computed from the modulus of elasticity of the concrete of the control cylinders. The agreement between the initial measured and calculated strains was generally close. The amount of creep for the six-year-old concrete is seen to be very small, ranging from 10 to 40 per cent of the elastic strain. This is in decided contrast to the creep for the columns loaded at 56 days, in which the creep after a year under load reached values varying from 330 to 460 per cent of the elastic strain. These curves confirm the statement that for concrete which has aged several years and is well dried out the creep will be almost negligible.

Discussion of the foregoing paper will be welcome if received in triplicate by the Secretary of the Institute by December 1, 1938. For such discussion as may develop readers are referred to the JOURNAL *for February 1939.*

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN FEBRUARY, 1939

Discussion of the paper by Richart and Heitman:

Tests of Reinforced Concrete Columns Under Sustained Loadings*

BY INGE LYSE†

The paper by Richart and Heitman presents some very important results from the A. C. I. Column Investigation. Since similar tests were made at Lehigh University it seems proper to present the results.

At Lehigh University twelve columns only of Series 3‡ were retained for testing beyond the one year period. These were cured 56 days in the moist room and then stored in the air of the laboratory. Eight of the columns were subjected to a sustained load corresponding to the maximum permitted by the A. C. I. 1928 building code, while four were stored under no load. Detailed information for these columns is given in Table 1 and Fig. 1 and 2. The deformation diagrams in Fig. 1 indicate that after about two years of sustained loading the columns have reached an equilibrium condition as far as plastic flow is concerned. Deformation changes after two years are primarily due to shrinkage variations. A seasonal variation is noted. During the winter months the relative humidity of the laboratory is low, consequently the shrinkage is high. In summertime the relative humidity is high, causing a decrease in the total shrinkage as shown in Fig. 1 for Columns 39 and 40, and 99 and 100.

The ultimate strengths of the columns at the age of about five years are given in Table 1 and are plotted in Fig. 2. The heavy broken lines of Fig. 2 give the strengths obtained on columns of Series 2** at the age of 56 days, the heavy solid lines the strengths of columns of Series 3 at one year, and the light lines the five-year strength of columns of Series 3. The solid light lines give the strengths of columns

*JOURNAL, Amer. Concrete Inst., Sept. 1938; *Proceedings* Vol. 35, p. 33.

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‡See Table 1, JOURNAL Amer. Concrete Inst., Jan. 1932; *Proceedings*, Vol. 28, p. 330.

**See "First Progress Report of Column Tests at Lehigh University," by W. A. Slater and Inge Lyse, JOURNAL, Amer. Concrete Inst., Feb. 1931; *Proceedings* Vol. 27, p. 677.

TABLE 1—STRENGTHS OF REINFORCED CONCRETE COLUMNS AT THE AGE OF FIVE YEARS

Column Number	Five Years Loading Condition	Reinforcement—Per Cent		Cylinder Strength p.s.i.	Column Strength lb.
		Longitudinal	Spiral		
33 & 34	Sustained	1½	1.2	3560	238,150
35 & 36	Sustained	6	1.2	3470	319,000
37 & 38	Sustained	1½	2.0	3690	266,900
39 & 40	None	1½	2.0	3460	289,150
97 & 98	Sustained	6	2.0	3910	353,900
99 & 100	None	6	2.0	3730	366,750

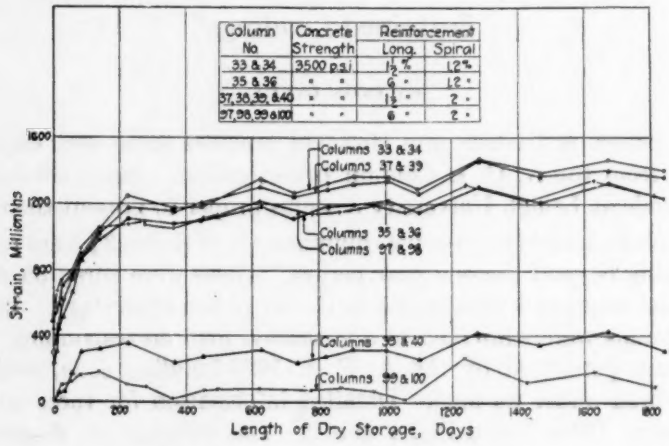
Column No
Concrete Strength
Reinforcement Long.
Spiral
33 & 34 3500 p.s.i. 1½ % 1.2%
35 & 36 " " 6 " 1.2 "
37,38,39, & 40 " " 1½ " 2 "
97,98,99 & 100 " " 6 " 2 "
Columns 33 & 34
Columns 37 & 39
Columns 35 & 36
Columns 97 & 98
Columns 39 & 40
Columns 99 & 100
Strain, Millionths
Length of Dry Storage, Days

FIG. 1—DEFORMATION OF COLUMNS DURING 5 YEARS OF DRY STORAGE

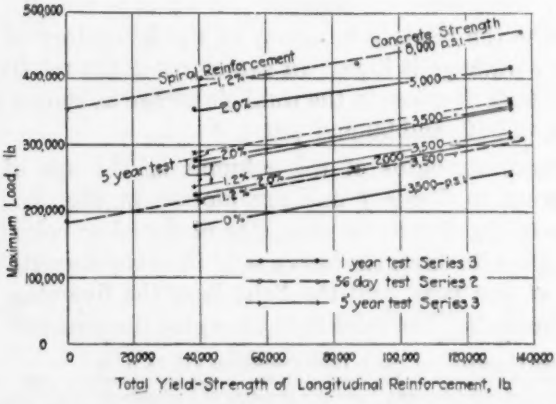

FIG. 2—RELATIONSHIP BETWEEN YIELD POINT STRENGTH OF REINFORCEMENT AND STRENGTH OF COLUMNS

which have been under sustained load for five years while the broken light line gives the results of the companion columns which were stored under no load.

It is noted both from Fig. 2 and Table 1 that the columns which had been under a sustained load for five years gave a strength slightly below that of companion columns stored for the same length of time under no load. However, the limited number of these columns and the slight difference in the results do not warrant any definite conclusion. Furthermore, the results at the University of Illinois show no effect on the six-year strength of columns. The maximum stress developed in the steel in the columns tested at Lehigh University was nearly 45,000 p.s.i., while at Illinois the steel stress did not exceed 30,000 p.s.i. for columns sustaining A. C. I. loads. Possibly the higher steel stresses account for the slight decrease in strength of the columns subjected to sustained loads at Lehigh University.

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Overvibration and Revibration of Concrete*

By Lewis H. Tuthill† and Harmer E. Davis‡
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SYNOPSIS

Since the rapid and widespread adoption of vibration as a means of placing concrete, no questions arise in the field more frequently concerning vibration than, "What is the harm from overvibration?" and "Does not vibration in most cases harm the concrete due to rupture of the initial set?" In this short paper these questions are discussed and answered for the benefit of the field man and others asking them.

Briefly, effects of overvibration are rarely encountered in concrete of medium and the drier consistencies. In the wetter consistencies horizontal stratification may result with detriment to durability.

Regarding revibration, actual tests show that as long as the concrete is not so old that sufficient vibration will not close, or "melt" it back together again, actual benefit, rather than harm, is the result of such treatment.

With the more frequent specification of vibration as a satisfactory means of compacting concrete in the forms, and with the more widespread use of vibration on the ordinary job, it seems appropriate that attention be given to some of the difficulties and dangers that arise in practice. Of the several faults in the technique of compacting concrete by vibration, one which merits particular consideration is overvibration,—which develops when the vibrator is allowed to remain in one place longer than is necessary just to consolidate thoroughly the mass within the range of influence of the vibrator. Consideration should also be given to the matter of revibration, by which is meant the application of vibration to a concrete that has already been compacted in place. The revibration of partially set concrete, or the vibration of reinforcing bars embedded in partially set concrete, has been considered by some engineers to be a harmful practice.

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Recent tests have shown, however, that under proper conditions, the strength of concrete may be considerably increased by revibration.

OVERVIBRATION

The danger and undesirability of overvibration, as well as the benefits of vibration, are primarily due to the peculiar effect of vibration on relatively stiff plastic masses of concrete. This effect is such that, during the period of vibration, the concrete has the characteristic of a liquid or of being of a very wet consistency. In this condition the force of gravity first draws mortar downward into voids and rock pockets, permits air to escape to the surface, and causes all solids and the concrete mass as a whole (within the influence of the vibrator) to seek the lowest possible elevation, all of which results in a more compact and a better consolidated portion of concrete.

Unless vibration is discontinued at this point, the undesirable results of overvibration become increasingly apparent. The force which at first produced the desired consolidation continues to operate. The concrete mass is not of uniform specific gravity; it is composed of water and numerous particles of solids of a wide range of sizes and having 2½ to 3 times the specific gravity of water. Although the mass acts as a homogeneous liquid during vibration, as soon as all general movement (void filling, consolidation, etc.) is completed there commences a gravimetric separation process. The larger particles of aggregate, having concentrated masses heavier than the surrounding momentarily liquid medium, gradually settle and, if vibration is continued long enough, will become concentrated at the bottom. On the other hand the finer fractions of the mortar move toward the surface, and become concentrated there and in the upper portion of the lift, with continued vibration. These finer fractions are of distinctly inferior quality as they are composed of mixtures of water with inactive and prehydrated cement particles, silt and clay from the sand.

It is these concentrations of inferior materials in the upper portion of the concrete lift that constitute the principal dangers of overvibration, and it should be emphasized that this undesirable effect is very rapidly increased as the consistency of the concrete is made wetter. This is another argument in favor of moderation in the use of mixing water when the concrete is to be vibrated.

An excellent quantitative example of this effect was unwittingly obtained when the first 6 x 12 in. cylinders of job concrete in tin cans were made on Colorado River Aqueduct tunnel lining work. For this work a consistency from a 5 to 6 in. slump was necessary with the methods used, a consistency readily susceptible to overvibration. In

an effort to protect the specimens from possible damage in the tunnel, they were placed on the motors and transported to the field laboratory at the batching plant by the first train leaving the tunnel after they were made. This procedure was promptly discontinued because the excessive vibration resulting therefrom completely reclassified the concrete in the 12-in. depth of the cylinder. Practically all aggregate larger than ¾-in. was in the bottom half of the cylinder. The material which had accumulated at the top was of such poor quality that it had to be removed before neat cement caps could be bonded to the specimens. Further, the water-cement ratio in the mortar in the top third of the specimen had been increased to such an extent that failure of the specimens in compression was principally in this portion of the cylinders.

Horizontal stratification of concrete lifts due to overvibration endangers the structure and reduces its quality because:

1. Uniformity and homogeneity is destroyed;
2. Owing to a water content which is higher than normal near the top of the lifts, there occur zones of weaker concrete not accurately measured by standard test cylinders;
3. The inferior concrete near the top of the lifts is more susceptible to weathering, particularly freezing and thawing, and it is more permeable;
4. Where the surface of overvibrated concrete forms a cold joint the bond is poor both chemically and mechanically. Such joints are susceptible to seepage, leakage, leaching and rapid weathering.
5. The "bleeding" effect is aggravated with the result that a layer of inferior material rises and becomes concentrated underneath each particle of aggregate thereby impairing its bond with the mortar and reducing the strength of the concrete.

Oftentimes, overvibration is a result less of surplus zeal for perfection than of some degree of lethargy on the part of the operator toward promptly removing the vibrator when it has performed sufficiently. Vibration is generally considered to be effectively complete when considerable amounts of air cease bubbling to the surface and when mortar just begins to flush to the top, or, as it has been expressed,[1] "when the concrete mass appears to have 'melted' to the point where the entire mix looks uniform." Time, however, is not the only factor involved; the consistency and proportions of the mix, the type of vibrator, the condition and shape of the forms and the distance between points at which the vibrator is applied,—all may have a bearing upon whether overvibration is a continual difficulty or

[1]R. F. Blanks and E. N. Vidal, "Placing Concrete by Vibratory Methods," in Papers presented at the Highway Conference at the University of Colorado, January 1937

an occasional occurrence. These matters have been touched upon in many of the papers on vibration in the last few years and are summarized in the recommendations of the Institute's Committee 609[2]. Apparently, however, they cannot be too much over-emphasized.

In many recent specifications which provide for the vibration of newly placed concrete, there is included a word of caution with respect to overvibration. The provisions of a specification, however, cannot cover the many varying conditions that are met with in the field in this regard, and it would seem that the judgment, watchfulness and skill of the field men must be relied upon in minimizing the occurrence of overvibration.

REVIBRATION

With regard to a study of revibration, relatively little has been done outside of a few exploratory tests to determine its effect upon strength of concrete and some observations of phenomena which accompany revibration. Such comments on these matters, however, may be of interest.

In considering the effects of revibration, it is desirable first to picture the structure of the cement paste during the setting process. Although the details of the process are still obscure, available evidence and authoritative opinion favor the following view[3]: soon after the mixing water begins to dissolve material at the surface of the cement grains, a colloidal gel is formed, gradually increasing in volume and consistency as further solution, hydrolysis and hydration take place; it is possible that initial set corresponds to the time when tentacle-like streamers of gel, growing from the individual grains of cement, meet and cause a rather rapid stiffening of the paste; it is possible, also, that there may be formed within the paste structure some crystalline products which aid in stiffening the partially set mass. During the setting process, there occurs a characteristic contraction or settlement of the mass; owing to friction between, and wedging of the aggregate particles in concrete, this tendency to contract is probably not fully satisfied, hence the application of external pressure may cause further consolidation.

Until the gel has attained considerable strength and rigidity, the concrete mass, when revibrated, can again be made to act as a fluid, although after several hours have elapsed from the time of mixing, considerable effort may be required. From observations on concretes revibrated after the lapse of various intervals from the time of mix-

[2] JOURNAL Am. Conc. Inst., Mar.-April, 1936; *Proceedings* Vol. 32, p. 445.
[3] Lynam, *Growth and Movement in Portland Cement Concrete*, Oxford Univ. Press, 1934.

ing, there appear to be three general stages as regards response to revibration.

In the first stage, while the mass slowly stiffens with time, the concrete is essentially plastic and is readily "fluidified" by vibration, although the time required to do this may be two or three times that required to consolidate the mass initially; in this stage, also, as the concrete is brought to the effectively fluid condition, a noticeable amount of air may be expelled from the mass.

In the second stage, which perhaps begins at about the time of initial set (considering the water-cement ratios such as are used in concretes), the effect of revibration appears at first to cause a slight disruption of the mass; continuation of the vibration, however, causes the mass slowly to "melt" together again, sometimes with the expulsion of considerable qualtities of free water. In relation to the setting process, it is possible that this behavior would indicate first a rupture of the filaments of gel which have begun to give definite structure to the stiff mass, and then a reduction of the gel to a viscous condition together with a further consolidation of the partially settled mass of particles. In this stage considerably more time and effort are required to reconsolidate the mass than required initially to compact the fresh concrete.

In the third stage the concrete is either so stiff that if disrupted, it is impossible to reconsolidate it effectively with ordinary vibratory equipment, or it is too stiff to be visibly affected.

In a limited number of tests, concretes in 6 by 12-in. cylinders have been successfully revibrated after intervals as long as 10 hours from the time of mixing. High frequency vibration was applied through the molds which were clamped to a specially constructed platform. As the result of revibration there were obtained increases in compressive strength of as much as 25 per cent over the strength of specimens which were not revibrated. The greatest increases in strength were obtained when the interval between time of mixing and time of revibration was of the order of 2 to 5 hours. It seems likely that the beneficial effect of revibration upon the compressive strength may be due to the reduction of voids which are developed within the concrete as the mass tends to settle.

In another test, two 6 ft. by 6 ft. by 4 in. wall panels were made and revibrated after intervals of 2 hr. and 10 hr. by means of a vibrator clamped to the forms. After the 2-hr. interval the concrete was successfully vibrated, but after the 10-hr. interval, small cracks were developed in the wall by the vibration which could not be closed again under the given conditions. This represents a case then of the

ill effect of revibration—the conditions set up by settlement of the concrete in the forms, coupled with vibration of the stiffened mass, produced ruptures which could not be healed by use of available power equipment.

A few tests were also made in which vibration was applied to plain round reinforcing bars cast vertically in 6 by 6-in. concrete cylinders. When the bars were revibrated 4 hr. after casting, it was observed that immediately after vibration was begun, bond was apparently broken and at the top a small annular space was visible between a bar and the surrounding concrete; soon, however, this space filled with paste and mortar, and some air was expelled from the viscous mass surrounding a bar. For bars which were revibrated, the bond strengths were from about 30 to 50 percent higher than the bond strengths for bars cast under similar conditions but not revibrated.

In another group of bond tests, deformed reinforcing bars were cast vertically in 6 by 6 in. cylinders and vibration was applied to the bars after intervals varying from 2 to 9 hours after time of casting. Both low and high frequency vibration was employed. Even with the low frequency vibration, no harmful effects of revibration were apparent for intervals up to about 6 hours. Compared with non-vibrated specimens, increases in bond strength at first slip of the order of 100 percent were obtained.

From the observations made during the tests outlined above, it would appear that as long as concrete remains in a fairly plastic state, say up to perhaps two hours after mixing, slight inadvertent revibration, such as may occur on the job, would cause no damage and possibly would result in some benefit. In a case in which a fresh lift is placed upon another, it is believed that by allowing the vibrator to penetrate a short distance into the lower lift, a degree of intermixing of the concrete near the boundaries of the two lifts may be obtained, with a better bonded and more homogeneous concrete as a result. In the case of steel protruding from a recently placed mass, it would seem that no damage would be done by contacting a bar with the vibrator, since in the plastic state, concrete can sustain large deformations without rupture.

With regard to revibration of concrete which has partially set, it would appear from the results of what tests have been made that substantial benefits may be derived if the operation is carried out under suitable conditions, i.e., those which will permit effectual reconsolidation. It is possible that practical advantage might be taken of this in the manufacture of precast products. On the ordinary construction job it does not appear feasible to use revibration where the

concrete has stiffened to any considerable extent, because the masses of concrete in place are so large that the time required to secure successful reconsolidation would make the operation impractical and because in intricate forms it would be impossible to determine whether complete reconsolidation had been secured.

No greater contribution to quality concrete construction has been made during this decade than that of the application of vibration to the compaction and consolidation of concrete. From these remarks suggesting caution in the extent and time of application of vibration, no measure of condemnation of this valuable procedure is intended or may be inferred. Recognizing its limitations and endeavoring to correct them at this stage of the development and use of vibration in concrete work, every opportunity should be taken by concrete engineers and others concerned to advance this important principle in placing concrete. If overvibration of wet concrete is limited and improper revibration of newly hardened concrete is avoided, the chances of harm from the use of the vibrator are exceedingly small and are vastly outweighed by the good work that can be accomplished with it.

Discussion of the foregoing paper will be welcome if received in triplicate by the Secretary of the Institute by December 1, 1938. For such discussion as may develop readers are referred to the JOURNAL *for February 1939.*

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . . . . SEPTEMBER, 1938

Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institution's Reviewers

Large workshop at Greenwich

E. R. BOLLEN, *Concrete and Constructional Engineering*. Vol. 33, No. 6, June 1938, p. 309-313.
Reviewed by GLENN MURPHY

Design and construction features of a 640-ft. by 69-ft. reinforced concrete workshop with a 20-ton overhead crane.

The design of box culverts

JOHN NAYLER, *Concrete and Constructional Engineering*., Vol. 33, No. 6, June 1938, p. 314-317.
Reviewed by GLENN MURPHY

Coefficients for determining moments in symmetrical rectangular box culverts subjected to concentrated, uniform or triangular loadings.

Reinforced concrete chimneys

C. PERCY TAYLOR and LESLIE TURNER, *Concrete and Constructional Engineering*, Vol. 33, No. 6, June 1938, p. 301-305. Reviewed by GLENN MURPHY

Third article in a series. This installment presents an emperical formula, and the data upon which it is based, for determining the temperature gradient in a lined chimney.

Calculation of stresses in reinforced concrete

Concrete and Constructional Engineering, Vol. 33, No. 6, June 1938, p. 299-300.
Reviewed by GLENN MURPHY

Editorial comments on the principal differences between the new Memorandum issued as a supplement to the Building By-laws of the London County Council and other bases of design, with reference to flat slabs and chimney shafts.

Recommendations for the control and reclamation of gullies

QUINCEY C. AYRES, *Bull. 121, Iowa Engineering Experiment Station*. Reviewed by H. J. GILKEY

Outlines the problem and describes and illustrates many types of small structures suitable for remedy and prevention. Timber, concrete, and stone constructions are shown for small dams and other regulating devices.

Electrification of the Wirral railway

Concrete and Constructional Engineering, Vol. 33, No. 5, May 1938, p. 249-257.
Reviewed by GLENN MURPHY

Design and construction features of several stations in the Wirral section of the L. M. S. R. Unusual methods were used for supporting forms for the rigid-frame foot bridges over the tracks and for the platform awnings.

Reconstruction of Leamington Spa station

Concrete and Constructional Engineering, Vol. 33, No. 5, May 1938, p. 258-264.
Reviewed by GLENN MURPHY

Progress report on reconstruction of old station buildings and platforms. The project includes building a subway, a 200-ft. retaining wall to support the platform and platform lines, and a horse-ramp. Description of the procedure of construction is given, including plans and photographs.

Treatment of plain surfaces in reinforced concrete construction

STATE CONSULTANT ERNST, *Beton und Eisen*, Vol. 37, No. 9, p. 145, May 5 and 20, 1938.
Reviewed by A. U. TREUER

This is an architectural and descriptive discussion of methods to obtain satisfactory concrete surfaces and of methods of pointing, crandalling, etc., subsequent to hardening, in order to obtain special esthetic effects.

Physical properties of typical American rocks

JOHN H. GRIFFITH (posthumusly published), *Bull. 131, Iowa Engineering Experiment Station*.
Reviewed by H. J. GILKEY

Reports determinations of physical properties of about 100 representative American rocks and attempts to correlate the strengths with such properties as density, absorption, and hardness. Considerable attention devoted to theoretical considerations. This bulletin is a fullow-up of Bull. 128 which deals with the thermal expansion properties of the same set of samples.

How to process soil-cement roads

FRANK T. SHEETS and MILES D. CATTON, *Engineering News Record*, Vol. 121, No. 1, July 7, 1938, pp. 20-24.
Reviewed by S. J. CHAMBERLIN

Recommended operations, equipment and field organization based on nearly thirty soil-cement jobs are given. Complete laboratory information on the soil is first necessary. On the job the operations are scarifying, pulverizing, mixing, careful addition of water, packing, shaping and rolling. Subgrades may be treated with small amounts of cement to reduce volume changes.

Old and new building methods

ROBERT W. ABBETT, *Engineering News-Record*, Vol. 121, No. 1, July 7, 1938, pp. 24-25.
Reviewed by S. J. CHAMBERLIN

Ancient and modern practices mingle on construction of concrete structure in Paris. All excavation was done by hand with pick and shovel, while the concrete

mixing was done with the most modern type of equipment. The concrete for the foundation was pumped through pipeline, but for the upper floors it was shoveled up through successive lifts as high as four floors.

Analysis of thin rectangular plates supported on opposite edges

D. L. Holl. *Bull. 129, Iowa Engineering Experiment Station.* Reviewed by H. J. Gilkey

Analysis is applicable to any homogeneous, isotropic plate relatively thin as compared to its lateral dimensions and the deflection of which is small relative to its thickness. The "thin" plate lies between the "thick" plate and the "membrane" in its behavior. The treatment is mathematical and methods developed can be extended to other cases. The results are applicable to reinforced concrete slabs.

The distribution of shearing stresses in concrete floor slabs under concentrated loads

M. G. Spangler, *Bull. 126, Iowa Engineering Experiment Station.* Reviewed by H. J. Gilkey

The experimental data secured are used as a basis for developing empirical working formulas for obtaining the effective widths of slab which resist shearing stresses under various forms of concentrated edge loadings. The studies included 20 different rectangular slabs ranging in thickness from 2½ to 6½ in.; in width from 5 to 7½ ft.; and in span from 3½ to 10 ft. All of the slabs were simply supported on two opposite edges and free on the other two edges.

A dry ice freezer

W. B. Ivie, *Engineering News-Record*, Vol. 120, No. 19, May 12, 1938, p. 682
Reviewed by R. W. Beal

Detailed description is given for the construction of a small, inexpensive unit for rapid freezing of concrete or concrete aggregates. The specimens are placed in a container, which is surrounded by kerosene and into which is placed dry ice. By alternating two sets of specimens and keeping the freezer working at all times it is possible to obtain one complete freezing-thawing cycle per hour on each of the two sets.

A contribution to the computation of beam grids (bridges)

Ernst Hennigs, *Beton und Eisen*, Vol. 37, No. 9, p. 151, May 5, 1938. Reviewed by A. U. Theuer

This article is an expansion of an earlier release by the author (*Beton und Eisen* Vol. 20, p. 323) in which a simplified method was proposed for computing the redistribution of moments due to cross ties connecting heavy outer beams with lighter interior ones. In the original article symmetrical loadings only were considered. The author here enlarges on his original paper by including cases of unsymmetrical loadings and unsymmetrical cross-sections and for any type of beam, simple, continuous, fixed, cantilever, etc.

Effect of dowel-bar misalignment across concrete pavement joints

ARTHUR R. SMITH and SANFORD W. BENHAM, *Proc. Am. Soc. C. E.*, Vol. 63, No. 6 (June 1937), pp. 1023-1034. Reviewed by H. J. GILKEY

Misalignment of dowel bars prevents free relative motion of adjacent slabs. Twelve of 14 installations were in 20 ft. pavement and 2 were in 10 ft. widening projects. The errors of 260 dowel bars were determined. Accuracy of placement is more important for expansion joints than for contraction joints and for thin slabs than for thick ones. Cracking and spalling of the slab occurs when misalignment exceeds permissible amounts.

Stresses and deflections in loaded rectangular plates on elastic foundations

GLENN MURPHY, *Bulletin 135, Iowa Engineering Experiment Station.* Reviewed by H. J. GILKEY

Presents a method for determination of stresses and deflections in rectangular plates supported on elastic foundations, with particular reference to plates which deflect free from the foundation under action of the load. Part I explains use of influence values for determinations at several points on the loaded plate, and Part II compares the analytical results with measured results from tests on steel plates on rubber foundations. Comparisons are also made with results from Westergaard's formulas for concrete pavement slabs.

Horizontal pressures on retaining walls due to concentrated surface loads

M. G. SPANGLER, *Bull. 140, Iowa Engineering Experiment Station.* Reviewed by H. J. GILKEY

Reports pressure measurements on walls 6 ft. high and 15 ft. long by means of stainless steel friction ribbons and Goldbeck pressure cells. Loads were applied with a heavily loaded truck. The rear wheels resting on a heavy timber parallel to the wall gave line loading when desired. Distributions were similar to those given by the classical Boussinesq equations but the magnitudes were much greater. More experimentation is needed to prove or disprove the new hypothesis advanced by the author.

Color analysis of portland cement concrete

W. M. DUNAGAN, *Bull. 139, Iowa Engineering Experiment Station.* Reviewed by H. J. GILKEY

Presents the results of an extended investigation of mineral pigments made by using the Razek-Mulder color analyzer. Color is expressed in terms of line (predominant wave length), intensity (whiteness), and chroma (purity). The effect of moisture conditions, curing, age, and other factors on the color characteristics was determined for tinted and untinted concrete. All concrete whitens or fades as favorable curing progresses due primarily to covering up by the release of the excess lime from the cement. The mineral colors do not themselves deteriorate. The author recommends using a sufficient excess of coloring pigment to leave the desired color after this whitening has occurred.

Earthquake stresses in an arch dam

IVAN M. MELIDOV and HAROLD E. VON BERGEN. *Proc. Am. Soc. C. E.*, Vol. 63, No. 10 (Dec. 1937), pp. 1851-1860. Reviewed by H. J. GILKEY

Basing the analysis upon Castigliano's Theorem of Least Work, the authors consider the cases of both fixed and hinged-end arches for: (a) accelerations which occur along the canyon, and (b) for accelerations across the canyon. From a typical calculation they conclude that stresses from accelerations along the canyon will seldom exceed 10 percent of the water-load stresses but that those across the canyon may be large. The smaller the central angle, the less vulnerable will be the arch against cross-wise acceleration. The vertical effect of the earth-quake is of minor importance.

Precast concrete riprap for Kingsley dam

D. A. BUZZELL, *Engineering News-Record*, Vol. 121, No. 2, July 14, 1938, pp. 54-55. Reviewed by S. J. CHAMBERLIN

A new type of reinforced concrete riprap has been designed for possible use to protect the upstream face of Kingsley Dam, Nebraska. The units are 4 ft. 6 in. long, 9 to 18 in. deep and 4½ to 6 in. thick. The lower courses would be anchored with ½-in. round asphalt dipped steel locking bars to the monolithic toe wall. The upper courses would be locked together with locking bars. Advantages claimed are: flexibility, for settlement; rough surface, to break up wave action; open enough to provide free drainage; close enough to conserve the fines in the fill; and structural stability.

Centrifugally-spun reinforced concrete pipes—plain and prestressed

KLEINLOGEL, *Beton und Eisen*, Vol. 37, No. 10, p. 161, May 20, 1938. Reviewed by A. U. THEUER

Twelve separate tests to determine the ability of centrifugally spun pipe to resist high internal pressures are described in detail. In conclusion the author summarizes the results of these tests by the statement that centrifugally-spun concrete pipes made by experienced and responsible firms may be depended upon without limitations to carry pressures up to 140 p. s. i. It is proposed in the next few years to use concrete pipes for a variety of purposes where the internal pressures will be of this order.

Highest multiple arch dam

Engineering News-Record, Vol. 121, No. 1, July 7, 1938, pp. 13-18 Reviewed by S. J. CHAMBERLIN

The Bartlett dam in Arizona is to be 750 ft. long, 287 ft. high, of ten arch barrel sections. Careful design studies showed that the stresses from temperature change will be considerably greater than those from water loads. Special, relatively intricate forms, and pumped concrete characterize the construction. The concrete, of low-heat cement, is vibrated to insure compaction and proper placement around the reinforcement. Conveyor belts carry the sand and coarse aggregates from bins to the mixer. The two-cylinder, 8-in. concrete pump was located on the canyon floor at first, and later moved upward. A suspension bridge, adjustable in height, carries the concrete line. Steel forms are being used to advantage.

Design of pile foundations

C. P. VETTER, *Proc. Am. Soc. C. E.*, Vol. 64, No. 2 (Feb. 1938) pp. 311-331.
Reviewed by H. J. GILKEY

Based upon a variety of simplifying assumptions, the author develops methods of design for transferring loads in any direction, including horizontal loads, to deep-lying foundation strata. Considerations from the point of view of soils mechanics have frankly been neglected in favor of those more purely of structural mechanics. The author believes that his developments are reasonably applicable to the majority of cases. The study was made in connection with the design of the Imperial Dam of the Boulder Canyon Project.

Expansion joints in concrete tanks

HENRY D. DEWELL, *Engineering News-Record*, Vol. 120, No. 19, May 12, 1938, pp. 687-688.
Reviewed by R. W. BEAL

An expansion joint design for floors and walls of concrete tanks is proving very satisfactory after many years of use. The joint consists essentially of a thin copper plate embedded in the concrete of one of two abutting slabs and floated in soft mastic packed in a recess formed in the concrete of the second slab. Observations on a total of 300 linear feet of this type of joint have established that the joints have definitely contracted and expanded without any observed leakage, and the concrete structures are nearly free from cracks. Details of the joint design are shown, together with a description of the different mastics tried and used and the provision made for replenishing the supply in the joints.

Novel concreting plant at Pilot Knob

Engineering News-Record, Vol. 120, No. 26, June 30, 1938, p. 896. Reviewed by R. W. BEAL

The Pilot Knob concrete wasteway on the All-American Canal, near Yuma, Arizona, is to be 100 ft. wide and will allow 10,000 sec.-ft. of water to be turned back to the Colorado. The feature of the batching plant was a layout of fan-shaped chutes resting on the sloping bank of the canal. There was a chute for each of four aggregate sizes. The fan-shaped construction permitted each chute to be used as a storage bin as well as for a supply chute. The mixing plant was located at the top of the wasteway, from which the concrete was moved in a bottom-dump hopper car on an inclined trestle railway. Concrete buggies were run under the hopper car, loaded, and wheeled directly to the forms.

Moments in flat slabs of various types

Progress Report of Subcommittee (F) of the Committee of the Structural Division on Masonry and Reinforced Concrete. *Proc. Am. Soc. C. E.*, Vol. 64, No. 3, (March 1938), pp. 529-537.
Reviewed by H. J. GILKEY

Supplements A. C. I. Bldg. Code of 1936 which applies only to: a series of rectangular slabs of approximately uniform size, arranged three or more rows of panels in each direction, and with capitals not smaller than 0.225 L (L = span length). This report presents coefficients for: panels arranged in two adjacent rows with C = .225 L (in either one or both directions), slabs without dropped panels in two or more rows (C = 0.15 L), and panels arranged in three rows the center one of which is a narrow corridor span (C = 0.15 L). In these developments extensive use has been made of the background work of Westergaard.

Paper in concrete roads

R. DITTRICH, *Betonstrasse*, (Road Abstracts), Vol. V, No. 1, (40). HIGHWAY RESEARCH ABSTRACTS

In the construction of the German motor roads, the use of building-paper between the foundation and the concrete is now obligatory. The paper need not be waterproof, but shall retain a minimum bursting strength of 3.55 p. s. i. over a circular surface 6 sq. in. in area after two hours immersion in water. The material shall weigh 4.4 to 5.3 ounces per sq. yd. and shall not form wrinkles in wet or windy weather. It should be supplied in rolls as wide as practicable. A minimum overlap of 2 in. at the edges and 8 in. at the ends shall be provided, the direction of the overlaps on grades, etc., being such as to prevent the entrance of water below the paper. Concreting should begin as soon as possible after the paper has been laid.

Making concrete for Conchas dam

Engineering News-Record, Vol. 120, No. 23, June 9, 1938, pp. 811-812. Reviewed by R. W. BEAL

Some of the most modern equipment available is being used at Conchas dam for handling the aggregates and mixed concrete. The special feature is a two-mile aerial tramway for transporting the aggregates from the wet-type aggregate processing plant to the batching and mixing plant at the damsite. The contractor's confidence in the dependability of the tramway is evidenced by the small storage bins at the mixing plant. Only storage for enough aggregate to supply the mixing plant for 8 hours is provided. The entire plant is electrified, and use is made of the most modern weighing and recording apparatus at the mixing plant, with many of the operations and controls being automatic. To date 60 per cent of the concrete has been placed and the equipment is proving very satisfactory.

Reinforced concrete girder bridges of over 100 ft. span

K. HAJNAL-KONYI, *Concrete and Constructional Engineering*, Vol. 33, No. 5, May 1938, p. 268-272.
Reviewed by GLENN MURPHY

Concluding article of a series begun in January. Brief discussion of Freyssinet's method and Dischinger's method of inducing initial stresses in steel and concrete. Summary of results of recent development: (1) Improvements in cement and use of vibrators permit higher working stresses; (2) pre-stressing steel and concrete increases safety against cracking; (3) welding steel reduces concrete dimensions; (4) Variation of the statical system during construction; (5) pre-loading scaffolding and piers to avoid settlement affecting completed structure; (6) use of solid slab sections and cellular sections instead of tee-beams; (7) further improvement could be made by using different factors of safety for dead and live load. Dead load is definite and does not require as high a factor of safety.

Approximate glass content of commercial portland cement clinker

WILLIAM LERCH, *Research Paper RP1066—National Bureau of Standards*, Vol. 20, Jan. 1938.
AUTHOR'S ABSTRACT

The approximate glass content was determined, by the heat of solution method, for commercial portland cement clinker obtained from 21 plants throughout the United States. The approximate glass content of these commercial clinker com-

positions was found to vary from 2 to 21 per cent. The approximate glass content was also determined for these clinker compositions after the clinker had been subjected to special heat treatment in the laboratory. The results indicate that for a given clinker composition, the heat of solution of the clinker, and likewise the glass content, are dependent on the cooling conditions to which the clinker has been subjected. Relatively high glass contents may be obtained by cooling the clinker rapidly, and relatively low glass contents by cooling it slowly.

Relative flexure factors for analyzing continuous structures

RALPH W. STEWART, *Proc. Am. Soc. C. E.*, Vol. 64, No. 1 (Jan. 1938) pp. 124-132.
Reviewed by H. J. GILKEY

Recognizing that the area of an $\frac{M}{EI}$ diagram represents the curvature of the part of the beam to which the $\frac{M}{EI}$ area is related, the author establishes as centers of rotation the points of intersection of the tangents to elastic curves. The use of the method differs from both the slope deflection and Maxwell-Mohr method of virtual work, in avoiding the necessity of solving groups of simultaneous equations. It differs from the end moment distribution by beginning the calculation at a point remote from the load and working toward it rather than away from it. It does in a "once-over" computation what end moment distribution does by a series of computations.

Cleaning concrete surfaces to ensure bond.

DANIEL MCFARLAND, *Engineering News-Record*, Vol. 120, No. 19, May 12, 1938, pp. 682-683.
Reviewed by R. W. BEAL

On the Grand Coulee dam many cleaning devices were tried in an effort to get the best possible bond between the concrete of successive pours (at minimum intervals of 72 hours). As soon as the concrete has set enough to stand it, the surface is cut with a jet of water and air to remove all loose particles which are not firmly a part of the concrete. The sprinkling water used for curing, washes out soluble material and forms a white deposit on the surface. It was found that this condition could be largely prevented by sprinkling sand on the concrete, as most of the white material leached out onto the sand instead of on the concrete surface. Since placing and removing sand proved rather expensive under certain conditions, the white deposit was sometimes allowed to form on the concrete and was then removed with a power-driven floor sander which had been fitted with steel wire bristles.

Minor constituents in portland cement clinker

HERBERT INSLEY and HOWARD F. MCMURDIE, *Research Paper RP1074—National Bureau of Standards*, Vol. 20, Feb. 1938. AUTHORS' ABSTRACT

The optical and structural properties of some minor phases in portland cement clinker as observed in thin sections, polished sections, and a new type of polished thin section are recorded. The similarity of such phases to those found in laboratory preparations is pointed out. Methods of determination of free CaO, free MgO,

"light" interstitial material, and "dark" interstitial material are given. The "light" interstitial material is shown to be $4CaO.Al_2O_3.Fe_2O_3$. There are three kinds of "dark" interstitial material which have the properties of: (1) glass; (2) $3CaO.Al_2O_3$; and (3) a prismatic crystalline material. The prismatic crystalline material apparently is a compound of the alkalies with some of the other oxides in the clinker. Glass is found in the majority of portland cement clinkers. At present, it is impossible to calculate precisely the compound composition from chemical analyses because of lack of knowledge of cooling conditions, the effect of minor constituents on phase relations, and other variables.

Expansion in autoclave

H. H. VAUGHN, *Rock Products*, June 1938, Vol. 41, No. 6, pp. 70-71.

Reviewed by R. N. YOUNG

A discussion of the expansion of portland cement is presented in the form of excerpts from a booklet published by Mr. Vaughn. Among the excerpts are the Summary and Conclusions. It is claimed that hard-burned MgO added to portland cement for the purpose of studying effects of naturally occurring MgO has led to misleading conclusions by other investigators, and that there is no evidence supporting the theory that MgO gives delayed expansion. Such expansion is caused by hard-burned free CaO that has remained unhydrated during the preparation of the specimen. The autoclave test is extremely sensitive to variations in the burning and cooling of clinker containing over 2 per cent MgO and gives high expansions in some cements which do not result in expansion in concrete structures. The LeChatelier test which is milder than the autoclave test does not have these faults but does detect unsatisfactory expansive properties. Therefore the LeChatelier test is preferred as a specification test. Discussion in great detail is given in support of these contentions.

Continuously varying moments of inertia in the analysis of statically indeterminate systems

GERHARD FISCHER, *Beton und Eisen*, Vol. 37, No. 7, p. 120, Apr. 5, 1938. Reviewed by A. U. THEUER

Author points out that in statically indeterminate systems long slender members are generally treated as if they had a constant moment of inertia even where it is known that this is not correct. The author concedes that the use of a mean value for the moment of inertia is sufficiently accurate for many purposes; however, in cases where the ends of a beam are either rigidly or elastically fixed large errors become possible.

To avoid the painstaking effort of calculated or nomographic solutions where the moment of inertia may be expressed by a simple mathematical function, the author sketches a method whereby general formulas or tables may be set up for practical use. The following shapes are included in the examples worked out: (1) frustrum of a pyramid; (2) beam of uniformly varying depth and constant breadth; (3) I-beam with uniform flange sections throughout its length, but uniformly varying depth of web; (4) beam consisting of two arms symmetrical over a support and of varying depth.

Comparative tests of beams reinforced with plain and with deformed bars

HENRYK GRIFFEL, *Beton und Eisen*, Vol. 37, No. 10, p. 169, May 10, 1938.
Reviewed by A. U. THEUER

A series of comparable beams, half of which were reinforced with plain bars and the other half with deformed bars, were tested in flexure over a span of 9.8 ft. under two point loadings. In cross section, these beams were 7.9 by 17.9 in. Six beams were singly and the other six doubly reinforced. Concrete having a compressive strength of 3500 p.s.i. was used in all cases except for two of the doubly reinforced beams where a 2750 p.s.i. concrete was substituted. The following quantities were measured in the case of each beam tested; (1) the deformation of the tensile steel at two points near mid-span, (2) the deformations of the concrete near mid-span, and (3) the deflections of the beam at mid-span and under the two supports. A complete record of the averages of these quantities for each type of beam tested is given in tabular form. The fracturing load of the singly reinforced beams using plain bars was 41,600 lb., and that using deformed bars 44,700 lb. This represents a ratio of 1 to 1.07. For the doubly reinforced beams this ratio was found to be 1 to 1.06 when 3500 p.s.i. concrete was used and 1 to 1.17 when 2750 p.s.i. concrete was used. In the future this work is to be extended to include beams with concretes of lower strengths.

Shrinkage and creep of concrete

H. SARTORIOUS-THALBORN, *Beton und Eisen*, Vol. 37, No. 13, p. 219, July 5, 1938.
Reviewed by A. U. THEUER

Author states that shrinkage without creep cannot take place in the case of concrete subjected to restraint by external forces or imbedded steel. In the case of axially reinforced elements shrinkage stresses reach a maximum value and then decrease with time until a state of equilibrium is established. The greater the percentage of reinforcement or restraint the sooner the maximum and constant shrinkage stresses are reached. For an average air-cured concrete with 1½, 4 and 6 per cent of axial reinforcement, the time required for the shrinkage stresses to become a maximum are 7½, 6 and 5 months, respectively. Multiplying these figures by 2½ will give the time required for equilibrium. Following the above order, maximum stress values induced in the concrete by these reinforcements will be 160, 280 and 340 p.s.i. Steel stresses decrease with increasing percentages of reinforcement. Maximum values will be 9900, 7100 and 5700 p.s.i. and equilibrium values 5700, 2800, and 2100 p.s.i. These values were arrived at by theory on the basis of data presented in the Reports of the A. C. I. Column Investigations made at the Universities of Illinois and Lehigh. They are advanced as reasonable average values for use as a guide in design. A knowledge of the order of these stresses and the factors that influence them are made a point of importance by the author.

Tests on reinforced concrete joints

J. BLEVOT, E. ENCONTRE, *Ann. De L'Institut Technique Du Batiment et des Travaux Publics*, 3'd year—No. 1, Jan.-Feb., 1938, p. 35-44.
Reviewed by P. H. BATES

The first part by J. Blevot contains a description of five compressive tests on Freyssinet type joints. It was necessary to support 193 tons (175 metric tons) on a hinged joint at the base of a column whose dimensions were 13.7 inches (35 cm) by 19.7 inches (50 cm). The joints dimensions were 5.9 inches (15 cm) by 19.7 inches

(50 cm). To resist the horizontal stresses a sheet iron shoe was placed on the bottom of the column and the ends of the horizontal reinforcing bars were welded to it. Four tests were made using this shoe and one without. The amount of reinforcing and its location were varied in the different tests, both in the column and in the concrete below the joint. As this type of problem is very difficult to analyze mathematically the data contained in this paper should be a valuable aid in the empirical solution of similar problems.

The second part by E. Encontre contains the description of a compressive test on a rocker joint of reinforced concrete. Similar joints were used to support one end of a 114 ft. (34.8 m) bridge. The rocker is a prism about 31 in. (80 cm) high and 20 in. (50 cm) in diameter. The bearing surfaces of the rocker (upper and lower) were each formed to a cylindrical surface of 47 in. (120 cm) radius. These rested on flat reinforced concrete surfaces. In the test a crack appeared when the load reached 352 tons (320 metric tons). The maximum load sustained was 880 tons (800 metric tons).

Dynamic investigation of concrete surfacing

A. Ramspeck, Berlin, (Road Abstracts), Vol. V, No. 1, (46). Highway Research Abstracts

Further measurements of the propagation of applied vibrations in concrete roads have confirmed that an intermediate layer of material between subgrade and surfacing affects the stresses set up in the concrete by vibration. Comparison with earlier measurements shows that not every kind of intermediate layer reduces the stresses. Thus a few inches of broken stone between concrete surfacing and subgrade considerably reduced the stresses, while a layer of sand 12 to 16 in. thick increased the stresses in the surfacing.

All tests have shown that with low frequencies the vibration of the surfacing conforms to that of the ground, but at higher frequencies, if the subsoil is poor, the surfacing is subjected to flexural vibration. The worse the subsoil, the lower the frequency at which flexural vibration becomes noticeable. So long as the surfacing vibrates in the same way as the subsoil the stresses in the surfacing are materially affected by the elastic behavior of the subsoil. When the surfacing undergoes flexural vibration, however, only the amplitude is affected by the subsoil or the intermediate layer: the stresses in the surfacing then depend only upon the dimensions of the surfacing and the properties of the concrete.

This research has been carried out for the Subsoil Research Group of the German Road Research Society by the German Soil Mechanics Research Association. Thus far it has been confined to cases where the machine generating the vibration was placed in the middle of a concrete slab. Investigations will be continued with the machine situated at the edge of a slab.

Solutions for certain rectangular slabs continuous over flexible supports

V. P. Jensen, *Univ. of Ill. Eng. Exp. Sta., Bul. 303*, 1938 (100 pages). Author's Abstract

Analytical results are given for particular cases of rectangular slabs carrying concentrated or uniform loads and supported by various combinations of flexible and rigid beams. The solutions were obtained as part of the work of an investigation made in cooperation with the United States Bureau of Public Roads and the Illinois

Division of Highways. The results given were obtained as solutions which satisfy Lagrange's differential equation of the deflected middle surface of the slab as well as the various conditions at the edges and interior supports.

Although the formulas are not suitable for direct use in design, a few illustrative curves and tables are given. It is contemplated that more of the equations will later be translated into numerical values and that the theoretical results will be interpreted in the light of test data. Certain formulas are capable of furnishing a means of evaulating, at least partially, the effect of continuity across the stringers or floor beams, the effect of flexibility of the supporting members, and the effect of diaphragms which are connected to the stringers but which are not themselves in contact with the slab.

Among the solutions obtained are those for the rectangular slab, simply supported on two opposite edges, and having either two or three longitudinal stringers. The bulletin is intended primarily as a source of reference from which material may be drawn for application to bridge slabs. Formulas are given for the bending moments in the supporting beams as well as in the slabs, the assumption being made that the beams exert only vertical forces upon the slabs.

A portable apparatus for measuring vibration in fresh concrete

GEORGE L. PIGMAN, FLOYD B. HORNIBROOK, and JESSE S. ROGERS, *Research Paper RP1101—National Bureau of Standards*, Vol. 20, May, 1938. AUTHOR'S ABSTRACT

A portable vibrograph was designed and constructed for measuring vibration in fresh concrete during the process of placing by vibration. An electromagnetic type of pickup unit, submerged with definite orientation within the concrete, picks up the vibrations and conveys an electric response through either of three electric circuits, and then through an amplifier to a cathode-ray oscillograph. Particle displacements, velocities, or accelerations can be measured and displacement, velocity, or acceleration wave forms obtained, depending upon the circuit chosen. Frequencies which form simple ratios with 60 cycles can be measured accurately, and other frequencies can be estimated.

The integrating or differentiating circuit-amplifier-oscillograph system was found to cause a variation of less than 3 per cent in the recording of a constant input for frequencies ranging from approximately 20 to 300 c/s.

The pickup unit was calibrated by use of microscope observations of amplitudes of vibration of a steel plate, vibrating in a horizontal direction at approximately 60 c/s, to which the pickup unit was attached. The amplitudes included in this calibration ranged from approximately 0.045 to 0.00035 in. Lower amplitudes were evaluated by extrapolation.

Calibrations of the acceleration and velocity scales were made by replacing the pickup unit with an audio-frequency oscillator supplying a sine wave of known frequency and voltage, corresponding to calibrated amplitudes. Accelerations ranging from approximately $\frac{1}{16}$ to 40 times gravity, and velocities ranging from 0.01 to 2.5 f/s were included in the calibration.

Typical curves are presented showing the variation of amplitude of vibration in concrete with distance from the vibrator. Photographs are given showing corresponding particle displacement, velocity, and acceleration wave forms produced by vibration of concrete with two different types of vibrators.

A new method of reinforcing large cross sections (girders)

ULRICH FISCHER, *Beton und Eisen*, Vol. 37, No. 12, p. 193, June 20, 1938. Reviewed by A. U. THEUER

In very heavily reinforced concrete members the placing of the steel becomes a problem. Heavy rods are hard to handle and tend to sag. Special supports are required for the steel and these usually remain in place after concreting. Incorporating steel in an unstressed and indifferent way, results in large tensile stresses in the concrete after the forms are removed and the member loaded. Cracking takes place subsequently. In small members this is seldom of importance but in large sections with complicated reinforcement this may give rise to serious consequences. In order to obviate these difficulties in the case of long span girder bridges where large steel concentrations are required, the author describes a new method of construction. Essentially the method consists of an independent load carrying reinforcing system which is progressively extended out from a pier and concreted as it is extended. Completed sections are subjected to stresses almost immediately after concreting instead of resting on a formwork until the entire structure is closed. The method is limited to elements such as beams, girders and arches, i. e., elements subjected to flexural stresses. In the case of girders it is pointed out that the stress trajectories may be considered as concentrated in tensile, compressive, and tie members. Certain general considerations, as well as a number of similar methods (Melan, Fryssinet, Dischinger, etc.) proposed or actually used in the construction of long span structures, are discussed with reference to the author's method.

Following this the author describes in detail how his method might be used for a 300 ft. span girder bridge. A diagram with numbered parts illustrating progressive states in construction is included.

Correlation of methods for measuring heat of hydration of cement

R. W. CARLSON and L. R. FORBRICH, *Industrial and Engineering Chemistry* (Analytical Edition) Vol. 10, No. 7, pp. 382-386, July 15, 1938. Reviewed by R. N. YOUNG

The limitation and advantages of three different types of calorimeters are pointed out and the results obtained by use of the several calorimeters are correlated. Sources of error and methods of correction are discussed. One of the discrepancies among results obtained from different calorimeters was attributed to immediate heat of hydration which is included in results obtained by the heat of solution method but is apt to be missed in others. Heat liberated up to 30 minutes varied among cements tested from 1.5 to 6.3 calories per gram, which is about 5 per cent of the average potential heat of hydration. The specific heat of concrete varies with temperature but largely on account of the change of specific heat of the hydrated cement content. Therefore error is introduced by assuming that the specific heat of well-hydrated cement or concrete applies at all ages. The heat of hydration increases with the water-cement ratio and although the effect is not great at 3 days, it is relatively important at later ages. The curing temperature has a marked effect on heat liberated at the early ages but little or no effect at later ages. Other sources of errors are in the drying of the sample and the carbonation of the hydrated sample. Each per cent CO_2 absorbed increases the heat of solution by 5.8 calories per gram.

On account of the errors involved in the determination of heat of hydration by the heat of solution method as employed by American laboratories, the results have been

too high to indicate correct temperature rise of concrete. Only by taking into account the various sources of error could identical and correct results be obtained by the three types of calorimeters.

Effect of stored merchandise on concrete

E. W. FRZISE, *Concrete and Constructional Engineering*, Vol. 33, No. 5, May 1938, p. 245-248. Reviewed by GLENN MURPHY

Results of observation made in Brazil on concrete structures well-built of sound materials. Cacao-beans produce acetic acid during fermentation. The acid combines with the lime components of the cement, resulting in a deep-seated disintegration which sometimes destroys the water-tightness of the structure. The action is accelerated by the presence of iron in the cement.

Coffee beans are removed from pulp and covering by washing and fermenting in tanks. Formic and lactic acids are produced and penetrate into the concrete through fine cracks and pores. Reacting with the lime and alumina components of the concrete, the acids form crystals which expand causing the spalling of layers as much as 1 in. thick. Where reinforcement is reached by the solution, wide-spread corrosion results.

Vegetable dyestuffs are produced from plants by fermentation. During the process glucose is produced, and it reacts with the cement, producing small granulations which are readily washed off. The bottom and walls of the tank are gradually worn away by the process, exposing the reinforcement. The rate of disintegration is less with well-cured concretes than with fresh concrete. Similar action occurs during fermentation or retting of stalks and leaves to produce fibers, and also during the processing of milled rags intended for paper.

Storage bins for ores and coal may suffer serious disintegration from sulfur acids produced by action of moisture on minerals. The only safe-guard is the use of cements very low in free lime.

Acetic and carbonic acids developed in the tanning process will cause disintegration if allowed to remain in contact with concrete

The Teufelstal-bridge on the through-highway Gera-Jena

OSKAR JUENLING, *Beton und Eisen*, Vol. 37, No. 11, p. 177, June 5, 1938. Reviewed by A. U. THEUER

The paper describes the preliminary planning and design and the actual construction of the Teufelstal bridge together with the investigative work carried out at the time. Being the longest concrete arch span in Germany, this bridge represents one of the major structures along the new German highways. The site may be briefly described as a narrow valley, 880 ft. across and 177 ft. deep with steep, rocky, symmetrical banks. The location being considered ideal for a concrete arch, no other type of bridge was considered for this site. Three designs were submitted in bid, (1) a hollow type and (2) a solid type arch, both with raised floor systems, and (3) a solid arch with floor resting directly on the crown of the arch. Two other types studied were a two-hinged arch and an arch with heavy crown tapering off toward the springings.

All five designs are shown photographically. Design 3 was adopted with certain dimensional changes as a result of model studies made to determine the esthetic suitability of the completed structure. As finally adopted, the bridge consists of two duplicate arch barrels 453 ft. in span and 85 ft. rise, 23 ft. wide, 9 ft. deep at the

springings and 4.2 ft. deep at the crown. The two barrels are separated by a 16 ft. clearance. The maximum stresses under the most unfavorable conditions of dead (shrinkage, creep, etc.) and live load are stated to be 1270 p.s.i. at the crown and 925 p.s.i. at the springings.

The supports for the floor system consist of solid walls carrying two main girders designed for maximum flexibility. The floor itself is anchored at the crown by means of anchor bolts and horizontal movement is provided for at either end of the floor system. A 6-in. layer of insulating material is included directly under the floor to lessen the tendency towards ice formation in winter.

The centering consisted of 6 wooden towers tied together and resting on tracks that permitted movement of the entire structure as a unit. The cement used was a low heat type and the concrete mix is described in detail. Deflections over the crown and quarter points were measured by means of stretched vertical wires. Rotation and displacement measurements at the springings were made, but no detectable movements were noted. Temperature measurements were also made using embedded thermo-couples. In addition to these field measurements, laboratory tests, which are still in progress, were commenced. A complete report of the investigation is promised for the future.

Autoclave tests and the microscope

L. S. Brown and M. A. Swayze, *Rock Products*, June 1938, Vol. 41, No. 6, pp. 65-69.
Reviewed by R. N. Young

This study was undertaken to establish the relationship of autoclave expansion of portland cement to expansion which might occur in concrete over a long period resulting from the hydration of free CaO and free MgO in the cement. The rate of hydration of these components in 70° F. water, the present steam test and the autoclave test were observed by use of the microscope. The results are well illustrated by photo-micrographs.

Free lime in cement occurs in three forms: light-burned (quick lime, finely porous), hard-burned (dense) and air-slacked (highly expanded). The authors state that the air-slacked lime is different from the normal crystalline $Ca(OH)_2$ and had not been previously identified. Some optical properties are given. Only the hard-burned form of MgO (dense crystalline particles) has been found in portland cement clinker. If the clinker is cooled quickly some MgO is retained in solution in glass. Under the conditions of a cement stored in 70° F. water for 24 hours, the maximum size particle of CaO which will fully hydrate, assuming exposure on all sides, is 4 to 5 microns. Under the same conditions no trace of hydration of MgO particles was found. When the cement was subjected to the A.S.T.M. steam test the maximum depth of hydration of CaO particles was about 20 microns. Some unexposed grains were not affected. No hydration of the MgO was detected. With the autoclave treatment all the free CaO was entirely hydrated. It appeared that MgO grains which were normal in size (not larger than 20 microns) were fully hydrated.

The second phase of the study consisted of comparisons of autoclave expansions with expansions obtained by long time water storage. A selected cement was used throughout these tests but the MgO content was varied with respect to grain size, form of MgO and proportion, by making the desired additions. It was shown that the autoclave will produce high expansions in cements due to an amount of crystalline MgO that would not be detected by long time water storage. Finer grinding of

cement reduces the expansive effect of MgO. Up to 5 per cent light-burned MgO did not increase the autoclave expansion. The tendency of a cement to expand excessively can be detected by the autoclave test long before the limits of free CaO and MgO are reached that are dangerous to the life of mortar or concrete in the field.

It was concluded that the A.S.T.M. steam test is inadequate for detecting the tendency toward delayed expansion, whereas the autoclave test, as adopted by the cement industry, is a reliable index for any unsoundness due to free CaO or MgO.

Joint spacing and dowelling (German motor roads)

R. DITTRICH, *Betonstrasse*, 1937, 12 (11) and (12). ROAD ABSTRACTS, London, Apr., 1938, No. 40. HIGHWAY RESEARCH ABSTRACTS

In a handbook issued in 1937, describing methods of concrete highway construction on the German motor roads, it is stated that joints shall be spaced 33 to 50 ft. apart. The lengths of successive slabs need not be varied. In mild equable climates, on stretches in flat country or in excavations (if foundation conditions permit), and in cold-weather construction, wide spacing is permissable, a narrower spacing being requisite if the annual temperature range is considerable. On high embankments, lower embankments of recent construction, on subgrades which are unstable or where the geological character varies considerably, in hot-weather construction or on curves of less than 330 ft. radius, the spacing may be further reduced to a minimum of 20 ft. where conditions are especially unfavorable, such as on approaches to large engineering structures. On bridge abutments the joints should be of the expansion type and should not be dowelled. The former practice of varying the length of road slabs has been abandoned because (a) the danger of resonance effects is negligible; (b) increased stability can now be ensured by use of dowelled joints. All joints are to be formed at right angles to the road axis, the possible utility of oblique joints becoming negligible at high speeds. Joints are to be continued across the full width of the pavement, since it was found that staggered joints may cause the formation of cracks.

As regards dowelling of joints, dowels shall be provided in all cases where frost-heaving or subsidence is likely to occur. The dowels must permit longitudinal movement of the slabs and transmit stresses across the joints without injury to the concrete. They must therefore be placed accurately in relation to the gradient and direction of the pavement, and midway through the thickness of the slab. Round dowel bars in expansion joints shall be at least 0.88 in. in diameter and 28 in. long; average spacing is 12 in., a closer spacing being adopted near the corners.

Dowels shall be connected by steel flats with blunted edges or supported by concrete blocks. The round end of each bar is left exposed, the tapered end being painted with a thin insulating coating of bitumen and protected by a cap which permits 0.8 in. expansion of the bar, packed with cork, shavings, etc. The holes in the joint inlays fit the dowelled bars very accurately.

Transverse dummy joints are dowelled in the same way as expansion joints, but the bars are 0.7 in. in diameter and are not capped. On curves of less than 2,000-ft. radius and on stretches where subgrade conditions are known to be unsatisfactory, anchor bars 5 ft. long and 0.56 to 0.64 in. in diameter must be provided across longitudinal or transverse expansion joints, in the middle third of the slabs, in order to permit longitudinal movement at both ends. The prescribed spacing is 30 in. Similar bars should be provided across the full width of expansion joints on stretches where

irregular subsidence or frost heaving are likely to occur; also in longitudinal dummy joints, along the full length of slabs at intervals of 5 ft.

Emphasis is placed on the necessity of providing proper support for dowel bars and for completing the whole joint assembly before it is placed in position.

A distribution procedure for the analysis of slabs continuous over flexible beams

N. M. NEWMARK, *Univ. of Ill. Eng. Exp. Sta., Bul. 304*, 1938, (120 pages). AUTHOR'S ABSTRACT

The author describes a method of analysis similar to moment distribution for certain types of continuous slabs subjected to concentrated or distributed loads. The method of analysis was developed as a part of an investigation of the effect of concentrated loads on reinforced concrete bridge slabs being carried on by the Engineering Experiment Station of the University of Illinois in cooperation with the United States Bureau of Public Roads and the Illinois Division of Highways. The procedure is not intended directly for use as a design procedure. The analysis applies to a structure with certain ideal properties, but does not deal with such problems as the ultimate strength of the slab, effects of temperature, shrinkage, and exposure to weather.

The method of analysis is applicable to any rectangular slab simply supported on two opposite edges, with any manner of support on the other two edges, and continuous over any number and spacing of rigid or flexible simple beams transverse to the simply supported edges. The slab may have sections of different depths or with different moduli of elasticity provided that the depth and elastic moduli are constant in each rectangular section formed by parts of the simply supported edges and transverse lines thereto. It is assumed that the supporting beams offer only vertical support to the slab, along a line. Under certain conditions the torsional restraint offered by the beams may be taken into account. The analysis is exact in the sense that it leads to numerical results in terms of infinite series that satisfy the fundamental equations of the ordinary theory of flexure of slabs for homogeneous, elastic, and isotropic material.

Numerical values of the quantities necessary for the analysis by the distribution procedure are tabulated. These quantities (stiffness, carry-over factors, fixed-end moments, and fixed-end reactions) are used in exactly the same way as in the analysis of continuous beams by moment distribution. The loading on the slab is divided into components of an infinite trigonometric series, and each component is treated separately. One obtains, finally, for each panel, component edge moments and edge deflections. The effects of these on the interior of the slab are considered as corrections to the effect of the load on a simply supported panel of the slab. The resultant moment, deflection, or other effect in the interior or at an edge is found by superposing all the component effects. In general the distribution procedure for any component of the loading converges more rapidly than the corresponding distribution in a continuous beam, but the procedure must usually be applied for several components in order to get the effect of the whole loading to the desired precision. Illustrative examples showing the way in which the procedure may be used, and the manner of obtaining the required numerical constants from the tables, are presented.

The distribution procedure may also be used to compute influence surfaces for a slab. One may estimate very readily, by rough calculations based on the distribution, the form of the influence surface for moment in a supporting beam. This may be used

to form some judgment of the way in which the load is "carried" to the simple supports in view of the static control for total bending moment in a cross-section of the structure. On any cross-section parallel to the simply supported edges through the entire structure consisting of a continuous slab and supporting beams, the magnitude of the total bending moment on the section is determined by statics. Part of the total moment is resisted by flexure in the beam, the remainder by flexure in the slab, but the manner in which the moment in the slab is distributed over the section, and the amount of the moment resisted by each beam, is statically indeterminate. One may, however, in many cases assign reasonable values to the magnitudes of the various moments. For loads great enough to cause cracking of the concrete, a rational analysis founded on good judgment is probably a better basis for design than an "exact" analysis developed on the assumption of a homogeneous, elastic material. The work in the bulletin is limited to a discussion of the latter case. However, as in many other problems, the "exact" analysis furnishes a basis for a better understanding of the behavior of the structure.

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Rigid Frame Bridges*

Report of Committee 314

HARDY CROSS, CHAIRMAN

SYNOPSIS

D. H. Pletta, (8) member of committee 314 has presented information as to rigid frame bridges already built. Here the committee presents its findings on controversial questions in the design of such structures.

It is desirable neither that the term "rigid frame bridge" be narrowly defined nor that the type of structure be standardized. The type seeks increase of clearance with important decrease in dead load by reducing center depth and also change from the conventional type of self-supporting abutment. Possible modifications involve true hinges at the center or at the footings and relief of the moment in the legs of single-span structures by counterweighting a cantilevered projection of the deck.

As a rough approximation, in single-span highway bridges, at the knee sixty per cent of the stress is due to dead load, thirty per cent to live load and ten per cent to shrinkage and change of temperature. At the center, stresses from these three sources are usually more nearly equal than they are at the knee. Other sources of stress are usually not important. As the number of spans increases, stresses near the knees of the outer span due to shrinkage and temperature change are much increased, (1) stresses from other sources are not greatly changed. The relative values here given may vary considerably.

Uncertainties as to physical properties are not more important here than in other concrete structures. They affect stresses at the center section and at the sections just above the footings more than at sections near the knees but failure through flexure at the center or at the footing does not necessarily imply failure of the structure as

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(1) See references at end of report.

a whole. It seems reasonable to use the same factor of safety on all sections; greater uncertainty as to stresses at crown and footing are compensated by less probability of failure from overstress.

Relative stiffnesses of members of the structure are influenced by variations in modulus of elasticity from one section to another, by reinforcement, and by cracks. Wilson and Kluge [2] indicate that the effect of these variations on shears, moments, and reactions are not significant [3]. Relative stiffness of members of different shape is uncertain, but this is usually less important here than in other structures of reinforced concrete. What is vaguely called "transverse distribution of concentrated loads" exists, as in other concrete bridges. Such auxiliary members as curbs and hand rails, through interaction with the main structure, affect the moments because of changes in relative stiffness; it is better to reduce such participation by pouring such members after forms are struck.

Valuable tests of Richart and Olson [4] on concrete specimens give no evidence that concentration of stress at sharp corners, as at the knees, affects the strength of the structure.

Expansion or contraction of three hundredths per cent due to temperature change, and shrinkage of two hundredths per cent, with additional provision for horizontal spread of footings up to one-half inch depending on judgment, have been assumed. The modulus to be assumed in such cases is often taken as 3,000,000 p.s.i. with an increase of one fourth in working stresses for combinations of the effects of gravity and of shrinkage and temperature change and of one third where foundation movement is also included.[5] Assumptions as to deformation modulus and working stress are clearly interrelated, practice varies as Pletta points out [6] and no canonical statement should be made. It has been suggested that danger from deformation strains are less than indicated by computed stresses [7] and there is reason to support the thesis. Plasticity is great when shrinkage is most active, and flow with time is a factor. Shrinkage effects may be reduced by construction procedure; in long structures expansion joints may be used.

A. G. Hayden has stated "we have..... found that, up to 80-ft. span at least, hinges are unnecessary and that construction may be simplified by omitting them.[8] His opinion is entitled to the highest respect and such definite evidence as exists appears to support it. Conditions of fixation against rotation at footings have little effect on the design. Special care should be taken in design, however, to reduce horizontal movement of footings.

A good deal has been written about methods of analysis for these structures; clearly the method is a matter of individual preference. Ordinary good judgment indicates that the possibilities that there may be freedom or that there may be restraint of horizontal movement of the deck ("side sway") should be considered. Horizontal earth pressures on abutments commonly affect the design little but the possibility of setting up passive pressures should be recognized.

The same specifications for loads, deformations and working stresses and the same precautions of good detailing should be used in the design of rigid frame bridges as are used for other structures of reinforced concrete; the assignment of the committee does not extend to standardization of general specifications.

REFERENCES

1) Maney in JOURNAL Amer. Concrete Inst., March-April, 1936; (*Proceedings* Vol. 32, p. 495) gives comparisons of stresses in single-span and multiple-span structures ("Analysis of Multiple-Span Rigid Frame Bridges by the Slope Deflection Method").

2) "Tests of Rigid Frame Bridges," Wilbur M. Wilson and Ralph W. Kluge, JOURNAL Amer. Concrete Inst., May-June, 1938; *Proceedings* Vol. 34, p. 625.

3) Interesting data are presented by Maney comparing results of analysis based on different assumptions, including variation of assumed stiffness at the knee. (See reference 1).

4) "Rapid and Long-Time Tests on Reinforced Concrete Knee Frames," F. E. Richart and T. A. Olson, JOURNAL Amer. Concrete Inst., March-April, 1937, *Proceedings*, Vol. 33, p. 459.

5) See, for example, the specifications for rigid frame bridges of the American Railway Engineering Association.

6) "Notes on Existing Rigid Frame Bridges in the United States," D. H. Pletta, JOURNAL Amer. Concrete Inst., March-April, 1938, *Proceedings* Vol. 34, p. 517.

7) "Effect of Time Yield in Concrete Upon Deformation Stress in a Reinforced Concrete Arch Bridge," W. M. Wilson and R. W. Kluge—Bulletin 275, Engineering Experiment Station, University of Illinois.

8) JOURNAL, Amer. Concrete Inst., Sept-Oct. 1935, *Proceedings* Vol. 32, p. 121.

Discussion, to close in April 1939* JOURNAL, *should reach A. C. I. Secretary in triplicate by Feb. 1.

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Discussion of the Report of Committee 314:

Rigid Frame Bridges*

BY CHARLES S. WHITNEY AND CLOSURE FOR THE COMMITTEE

BY CHARLES S. WHITNEY†

The writer wishes to take exception to the statement "Valuable tests of Richart and Olson on concrete specimens give no evidence that concentration of stress at sharp corners, as at the knees, affects the strength of the structure" unless it is limited to cases where ordinary percentages of steel are used. Quoting from a recent paper:‡

Attention should be called to the test of knee frame No. 19 given in Table III. At the suggestion of the writer two frames with about 3 per cent of tension steel were tested and the breaking load was found to be about 80 per cent of over-reinforced straight beams and of columns which failed in compression. Before failure, cracks formed parallel to the compression face at the knee, and the writer suggests that the failure at low load may be due to excessively high shearing stresses caused by the sharp curvature of the line of compression stress around the angle of the knee. There may be similar action at the junction of frame members, and it appears that further information is needed about the actual strength in compression or shear at such angles. It is the writer's opinion that all other frames in Table III failed primarily in tension and give no indication of the compressive strength. They indicate no weakness in compression due to the concentration of stress at the knee when the ordinary amount of reinforcement is used, but there is not sufficient information to determine the upper limit of reinforcement for such cases. It is possible that special shear reinforcement is needed.

Current practice provides a much higher factor of safety against compression failure in the concrete than against tension failure in the steel. If sufficient steel is provided the ultimate resisting moment of a straight beam is given closely by the expression $M = \frac{bd^2 f'_c}{3}$ with a factor of safety of two and one half, consistent with stresses of

*JOURNAL, Amer. Concrete Inst., Nov. 1938; *Proceedings* Vol. 35, p. 69.
†Consulting Engineer, Milwaukee, Wis.
‡"Eccentrically Loaded Reinforced Concrete Columns" by Charles S. Whitney, *Concrete and Constructional Engineering*, London, Nov. 1938, p. 8.

18,000 or 20,000 p.s.i. in the steel, this would permit about twice as high a bending moment in a rectangular beam than allowed in current practice. With the present high strength and reliability of concrete, there are therefore interesting possibilities for the more efficient use of the material and the design of lighter and more slender structures. Increased slenderness and flexibility will result in reduced stresses due to shrinkage and temperature changes thereby permitting the construction of longer and more economical continuous frames.

Tests made to date indicate that in knees with higher percentage of steel there may be a reduction in the compressive resistance moment as compared with that of straight beams or columns. Sufficient tests have not yet been made to determine the cause or the amount of such reduction and the writer hopes that a thorough investigation will be made, because it is of great importance to the development of elastic frame construction.

COMMITTEE'S CLOSURE

The discussion by Mr. Whitney raises the question of increasing compressive flexural stresses permissible in reinforced concrete designs. This proposal needs full discussion; probably none will insist more strongly than Mr. Whitney that it be discussed carefully and fully. The committee feels that consideration of so broad a question in a report devoted to rigid frame bridges would be injudicious. Some doubt seems permissible whether the data presented by Richart and Olson in their closing discussion should be taken as evidence of weakening due to the sharp corner where large percentages of tensile steel are used; if such weakening is shown, it would, if allowable compressvie stresses are increased, need to be considered in designing structures other than those discussed in our report. The question does not, however, seem significant in considering the design of rigid frame bridges under existing general specifications.

The statement quoted from the committee report by Mr. Whitney was intended to allay anxiety induced by injudicious interpretation of analyses made by protoelasticity or by the mathematical theory of elasticity; the committee does not feel that the statement needs modification.

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A Short-Time Test for Effect of Type of Cement on Concrete Shrinkage*

By G. E. Troxell†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

Some cements produce a large drying shrinkage of the concrete in which they are incorporated; others produce a much smaller shrinkage, in concretes identical except for the cements used. The lower the drying shrinkage the less the tendency of a concrete structure to crack. The designer of a structure is concerned with the shrinkage to be expected and should introduce enough contraction joints to prevent cracking which is not only unsightly but also endangers the stability of the structure. The designer may even desire to specify a maximum limit on the concrete shrinkage. Normally, the effect of the type of cement on shrinkage is determined by prolonged tests. This paper describes a procedure using short-time test results on 3 by 6-in. concrete cylinders and on 1½ by 1½ by 12-in. mortar bars from which the long-time shrinkage of concrete can be predicted with reasonable accuracy.

INTRODUCTION

Frequently there arises a question concerning the relative drying shrinkages of concretes containing cements of different compositions and fineness. Either the question is answered by the results of prolonged tests or it is left unanswered due to the length of time considered necessary to obtain satisfactory results. In the hope of developing a short-cut procedure for the determination of the effect of the type of cement on the shrinkage of concrete, investigations have been in progress in the Engineering Materials Laboratory of the University of California for the last two years.

Shrinkage of concrete results primarily from loss of moisture. Since the drying process is ordinarily slow, it was thought that either by using accelerated curing and drying conditions or by decreasing

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the size of aggregate and specimen and the ratio of aggregate to cement, some reliable measure of long-time concrete shrinkage could be obtained in a short period. To this end, the tests were designed to determine the ratio between the long-time shrinkage of a concrete specimen and the short-time shrinkages at different ages of the same specimen or of other types of specimens suitable for more rapid drying. To be of value, such a ratio for any given short shrinkage period should be essentially constant for all cements.

The general value of a short-time test has been shown in an earlier paper* where it is stated in connection with tests of mortar bars only, "It appears that the relative differences in contraction as between cements develop during a rapid first stage of drying, after which these differences remain in relative proportion during a secondary, slower stage of drying." As these earlier tests of small mortar bars were not tied in with shrinkage tests of corresponding concretes, the present program was initiated, in part, to establish their relationship.

MATERIALS AND TYPES OF SPECIMENS

Properties of Cements—To insure conformity of any cement normally encountered with the results of this series of tests, eight cements having a wide range in composition and fineness were included in the investigation. A brief description of the eight cements is presented in Table 1, and chemical and physical properties in Table 2.

Specimens for Long-Time Tests—Since length changes of 3 by 6-in. concrete cylinders are commonly used in several laboratories for determinations of volume changes in concrete, concrete cylinders of this size having a cement-aggregate ratio of 1:5.6 mix by weight and with aggregate of maximum size ¾ in. were used for all long-time shrinkage observations. As the shrinkage of concrete is affected by its consistency, the concrete for each of the eight cements was gaged to a fixed flow of 60 per cent, determined in accordance with A. S. T. M. Designation C124-36T. The shrinkage of such specimens after 7 or 28-days curing and then drying for 200 days in air of 50 per cent relative humidity at 70°F. was taken as the long-time shrinkage.

Specimens for Short-Time Tests—The following types of specimens were subjected to short-time tests:

1. 3 by 6-in. cylinders used for the long-time shrinkage observations also were measured frequently at early ages to determine their short-time shrinkages.
2. 1½ by 1½ by 12-in. bars cast from the same batch of concrete as the 3 by 6-in. cylinders.

*"Properties of Mortars and Concretes Containing Portland-Puzzolan Cements," JOURNAL, Amer. Concrete Inst., Sept.-Oct., 1935; *Proceedings*, vol. 32, p. 80 (see p. 107).

TABLE 1—DESCRIPTION OF CEMENTS

Designation	Description
C1	Normal cement (specific surface 1600 sq. cm. per g.).
C2	Low heat cement (low lime).
C3	High early strength cement (high lime).
C4	Modified cement.
C64	Commercial portland-puzzolan cement (75 per cent modified cement, 25 per cent calcined volcanic sand).
C129	Cement C1 ground to a specific surface of 1980 sq. cm. per g.
C4P4	80 per cent of cement C4 plus 20 per cent of ground Potomac boiler slag, a puzzolanic material.
C4P9	80 per cent of cement C4 plus 20 per cent of Indiana fly ash, a puzzolanic material.

TABLE 2—CHEMICAL AND PHYSICAL PROPERTIES OF CEMENTS

Properties of Cements		Cement							
		C1	C2	C3	C4	C64	C129	C4P4	C4P9
Compound Composition of Clinker, per cent	C_3S	53	42	60	50	54*	Same	50*	50*
	C_2S	23	34	15	25	18*	as	25*	25*
	C_3A	12	4	8	6	5*	Cement	6*	6*
	C_4AF	8	15	13	15	18*	C1	15*	15*
Fineness, sq. cm. per g.		1590	1600	2220	1600	1970	1980	1770	1800
Normal Consistency, % water by weight.		22.5	21.5	23.4	22.5	24.5	....	22.0	22.8
Time of Setting, Gillmore, hr: min.	Initial	2:20	3:00	1:15	2:15	1:55	....	2:25	3:10
	Final	3:40	4:45	2:05	4:00	3:20	....	3:50	5:00
28-Day Tensile Strength of Std. Briquets, p.s.i.		359	355	451	367	355	....	363	388

*For portland cement constituent of portland-puzzolan cement.

3. 1½ by 1½ by 12-in. bars of 1:3.25 mortar with graded local sand having a fineness modulus of 2.60. The water-cement ratios for these mortar bars were the same as for the corresponding concretes.

4. ½ by 6-in. cylindrical pencils of 1:3 mortar; the aggregate consisted of 50 per cent mine run and 50 per cent standard Ottawa sand. All mortars for the pencils were made to give a uniform flow of 50 per cent using a 4-in. cone and thirty ⅛-in. drops as specified in A. S. T. M. Designation C87-36.

5. ½ by 6-in. cylindrical pencils of neat cement. The neat cement pastes were of such consistency that they flowed from a 100-ml. pipette (having a 5-mm. bore outlet tube 15 cm. long) at a rate of 3 ml. per second when under a pressure of 1 p.s.i.

Water-Cement Ratios—The water-cement ratios for the various types of specimens are presented in columns 2, 3, and 4 of Table 3. It is evident from these values that the water-cement ratios for the concrete, mortar, and neat paste mixes of a given cement do not have the same relative values for the eight cements. Additional irregularity appears in the values of mortar flow presented in column 5. Although the mixes for the mortar bars had the same water-cement ratios shown in column 2 for concretes of equal consistency (60 per cent flow), it will be noted that the mortar flows shown in column 5 decidedly are not equal nor are they proportional to the water-cement ratios of column 2. These relationships of water-cement ratio and consistency have an important bearing on the study of shrinkage due to the influence of water content of the mix.

TABLE 3—WATER-CEMENT RATIO FOR ALL MIXES AND FLOW OF MORTAR FOR BARS

Cement	Water-Cement Ratio, by weight			*Flow of Mortar for Bars, %
	Concrete Cylinders and Bars and Mortar Bars	Mortar Pencils	Neat Cement Pencils	
C1	0.426	0.430	0.375	34
C2	0.430	0.425	0.368	33
C3	0.429	0.435	0.420	33
C4	0.434	0.435	0.390	30
C64	0.440	0.430	0.435	35
C129	0.414	0.420	0.395	35
C4P4	0.430	0.434	0.390	25
C4P9	0.429	0.435	0.420	33

*Flow of mortar having water-cement ratio for corresponding concrete.

Curing and Storage Conditions—The curing and storage conditions for the several series of tests are presented in Table 4. For series A to E inclusive one group of three specimens of each cement was cured 7 days in fog at 70°F., while another similar group was cured under the same conditions for 28 days. For series B to E inclusive, an additional curing period of 60 days was used for the four slower hydrating cements C2, C4, C64, and C4P9. At the close of the curing periods, the specimens of series A to E inclusive were transferred to storage in air maintained at 50 per cent relative humidity and 70°F. They were stored under these conditions continuously.

In addition to the regular treatments mentioned above, three other series of specimens, F, G, and H, were subjected to the conditions shown in Table 4. These were accelerated curing and drying conditions, designed to produce in a 28-day period, either a large proportional part of, or the total long-time shrinkage which would occur under normal drying conditions.

Shrinkage Observations—For observations of shrinkage, stainless steel plugs were cast in the ends of the 3 by 6-in. cylinders and the ½ by 6-in. pencils. A dial gage comparator was used to measure the overall length between these end plugs. For the 1½ by 1½ by 12-in. bars, stainless steel plugs were cast in two opposite faces on 10-in. centers for observations of length change by means of a strain gage.

All specimens of series A to E inclusive were measured (1) when removed from their molds at the age of 1 day, (2) at the end of the curing period, and (3) at frequent intervals during the dry storage period. The specimens of the accelerated series F, G, and H were measured as in (1) and (2) above and finally at the age of 28 days, at the end of the accelerated drying period.

TEST RESULTS

Long-Time Concrete Shrinkage—The gross shrinkage of the concrete cylinders when dried for 200 days, after standard curing for 7

TABLE 4—CURING AND STORAGE CONDITIONS

Series	Specimen Type	Curing Conditions	Storage Conditions**
A	3 by 6-in. concrete cylinders	7 and 28 days in fog at 70°F.	Dry air
B	1½ by 1½ by 12-in. concrete bars	7 and 28 days in fog at 70°F.*	Dry air
C	1½ by 1½ by 12-in. mortar bars	7 and 28 days in fog at 70°F.*	Dry air
D	½ by 6-in. cyl. mortar pencils	7 and 28 days in fog at 70°F.*	Dry air
E	½ by 6-in. cyl. cement pencils	7 and 28 days in fog at 70°F.*	Dry air
F	½ by 6-in. cyl. cement pencils	24 hr. in mold at 70°F., 5 hr. in steam at 212°F., 19 hr. in fog at 70°F.	26 days in dry air
G	½ by 6-in. cyl. cement pencils	24 hr. in mold at 70°F., 20 days in fog at 70°F	4 days in dry air, 2 days in oven at 230°F., 1 day in desiccator.
H	½ by 6-in. cyl. cement pencils	24 hr. in mold at 70°F., 48 hr. in fog at 70°F., 18 days in water at 100°F.	4 days in dry air, 2 days in oven at 230°F., 1 day in desiccator

*Specimens of cements C2, C4, C64 and C4P9 also were subjected to 60-day curing.
**Dry air was maintained at 50 per cent relative humidity and 70°F.

or 28 days, are presented in Table 5. A wide divergence of values for the eight cements will be noted.

TABLE 5—GROSS SHRINKAGE OF CONCRETE CYLINDERS DRIED 200 DAYS

Curing Period, days	Unit Shrinkage, millionths							
	C1	C2	C3	C4	C64	C129	C4P4	C4P9
7	460	510	470	580	630	470	590	570
28	440	510	460	540	560	420	475	475

Specimens were 3 by 6-in. concrete cylinders of 1:5.6 mix by weight having 60 per cent flow.

Shrinkage Ratios—The shrinkages of all specimens subjected to normal drying in air at 70°F. and 50 per cent relative humidity were expressed as a percentage of the long-time shrinkage of the corresponding 3 by 6-in. concrete cylinders dried for 200 days after both 7 and 28-day curing of the cylinders. Table 6 presents the short-time shrinkage values for concrete cylinders, concrete bars and mortar bars cured both 7 and 28 days and after drying periods of 14, 28, 42 and 56 days, all values being expressed as a percentage of the gross long-time shrinkage of concrete cylinders standard-cured 7 days and dried 200 days. Due to the small size of some of the short-time test specimens and their resultant rapidity of drying, it was considered that the degree of hydration of the cement in such specimens would be somewhat less than for the cylinders. Hence the shrinkage values for both the 7 and 28-day curing of the short-time test specimens compared with both the 7 and 28-day curing of the long-time test cylinders were computed. Due to lack of space, only a few of the shrinkage values are included in Table 6. In general, specimens of a given type subjected to identical conditions of curing and drying, show somewhat similar shrinkage percentages or shrinkage values.

TABLE 3—WATER-CEMENT RATIO FOR ALL MIXES AND FLOW OF MORTAR FOR BARS

Cement	Water-Cement Ratio, by weight			*Flow of Mortar for Bars, %
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Shrinkage Observations—For observations of shrinkage, stainless steel plugs were cast in the ends of the 3 by 6-in. cylinders and the ½ by 6-in. pencils. A dial gage comparator was used to measure the overall length between these end plugs. For the 1½ by 1½ by 12-in. bars, stainless steel plugs were cast in two opposite faces on 10-in. centers for observations of length change by means of a strain gage.

All specimens of series A to E inclusive were measured (1) when removed from their molds at the age of 1 day, (2) at the end of the curing period, and (3) at frequent intervals during the dry storage period. The specimens of the accelerated series F, G, and H were measured as in (1) and (2) above and finally at the age of 28 days, at the end of the accelerated drying period.

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*Specimens of cements C2, C4, C64 and C4P9 also were subjected to 60-day curing.
**Dry air was maintained at 50 per cent relative humidity and 70°F.

or 28 days, are presented in Table 5. A wide divergence of values for the eight cements will be noted.

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Specimens were 3 by 6-in. concrete cylinders of 1:5.6 mix by weight having 60 per cent flow.

Shrinkage Ratios—The shrinkages of all specimens subjected to normal drying in air at 70°F. and 50 per cent relative humidity were expressed as a percentage of the long-time shrinkage of the corresponding 3 by 6-in. concrete cylinders dried for 200 days after both 7 and 28-day curing of the cylinders. Table 6 presents the short-time shrinkage values for concrete cylinders, concrete bars and mortar bars cured both 7 and 28 days and after drying periods of 14, 28, 42 and 56 days, all values being expressed as a percentage of the gross long-time shrinkage of concrete cylinders standard-cured 7 days and dried 200 days. Due to the small size of some of the short-time test specimens and their resultant rapidity of drying, it was considered that the degree of hydration of the cement in such specimens would be somewhat less than for the cylinders. Hence the shrinkage values for both the 7 and 28-day curing of the short-time test specimens compared with both the 7 and 28-day curing of the long-time test cylinders were computed. Due to lack of space, only a few of the shrinkage values are included in Table 6. In general, specimens of a given type subjected to identical conditions of curing and drying, show somewhat similar shrinkage percentages or shrinkage values.

TABLE 6—SHRINKAGE EXPRESSED AS PERCENTAGE OF LONG-TIME SHRINKAGE OF CONCRETE CYLINDERS CURED 7 DAYS AND DRIED 200 DAYS

Drying Period, days[1]	Type of Specimens[2]	Curing Period, days	Percentage of Long-Time Concrete Shrinkage							
			C1	C2	C3	C4	C64	C129	C4P4	C4P9
14	Concrete Cylinders	7	59	48	52	57	57	57	64	52
		28	62	55	61	52	53	55	49	49
	Concrete Bars	7	70	70	71	70	73	64	72	71
		28	65	68	70	59	60	64	61	63
	Mortar Bars	7	122	112	119	128	141	127	128	125
		28	126	129	116	99	122	119	80	112
28	Concrete Cylinders	7	76	66	69	72	76	77	81	71
		28	75	71	77	66	65	68	61	62
	Concrete Bars	7	83	88	86	78	83	74	83	88
		28	74	80	80	68	67	72	72	72
	Mortar Bars	7	152	143	145	150	159	153	149	150
		28	149	178	141	123	147	142	106	139
42	Concrete Cylinders	7	78	76	79	78	84	84	86	79
		28	83	81	81	87	71	78	66	69
	Concrete Bars	7	88	96	93	81	87	81	86	96
		28	78	86	84	69	71	81	77	76
	Mortar Bars	7	165	159	157	159	166	168	159	161
		28	158	193	156	134	156	153	115	150
56	Concrete Cylinders	7	87	83	83	83	89	88	87	86
		28	87	87	85	76	76	79	71	74
	Concrete Bars	7	90	100	97	85	89	82	87	99
		28	82	89	88	75	72	76	78	79
	Mortar Bars	7	172	163	165	166	170	172	166	168
		28	163	198	164	139	162	159	120	156

(1) All specimens standard cured for periods noted, then dried in air of 50 per cent relative humidity at 70°F.

(2) Concrete cylinders were 3 by 6-in. and of 1:5.6 mix by weight having 60 per cent flow.
Concrete bars were 1½ by 1½ by 12-in. and of same mix as concrete cylinders.
Mortar bars were 1½ by 1½ by 12-in. and of 1:3.25 mix by weight having the same water-cement ratios as for the corresponding concretes.

The similarity of these values is greatest when both the long-time test cylinders and their companion short-time test specimens are cured 7 days. For both concrete cylinders and mortar bars the values become more uniform with increasing periods of drying.

Table 7 summarizes the average shrinkage values for all cements for specimens cured 7 days.

Errors in Prediction of Long-Time Concrete Shrinkage—The general agreement of the individual shrinkage values of Table 6 is shown in Table 8 in which is summarized the maximum and the average percentages of error that would have resulted in the prediction of the long-time concrete shrinkages from the short-time shrinkages and the average shrinkage percentages given in Table 7. Except for the concrete bars, the errors show a progressive decrease with an increase in time of drying. In general, the values are reasonably satisfactory; it appears that 1½ by 1½ by 12-in. mortar bars dried for 28 days will give long-time shrinkage values reliable within about 6 per cent

TABLE 7—AVERAGE SHRINKAGE EXPRESSED AS PERCENTAGE OF LONG-TIME SHRINKAGE OF CONCRETE CYLINDERS

Drying Period, days[1]	Average Percentage of Long-Time Concrete Shrinkage		
	Concrete Cylinders[2]	Concrete Bars[2]	Mortar Bars[2]
14	56	70	127
28	73	85	150
42	81	88	162
56	86	91	168

(1) All specimens standard cured for 7 days, then dried in air of 50 per cent relative humidity at 70°F.

(2) Concrete cylinders were 3 by 6-in. and of 1:5.6 mix by weight having 60 per cent flow. Concrete bars were 1½ by 1½ by 12-in. and of same mix as concrete cylinders. Mortar bars were 1½ by 1½ by 12-in. and of 1:3.25 mix by weight having the same water-cement ratios as for the corresponding concretes.

TABLE 8—ERROR IN PREDICTION OF LONG-TIME CONCRETE SHRINKAGE FROM SHORT-TIME TEST

Drying Period, days[1]	Percentage of Error in Prediction					
	Concrete Cylinders[2]		Concrete Bars[2]		Mortar Bars[2]	
	Maximum	Average	Maximum	Average	Maximum	Average
14	14	7	9	2	11	5
28	11	5	13	5	6	2
42	7	4	8	5	4	2
56	4	2	10	6	3	2

(1) (2) See footnotes (1) (2) Table 7 above.

in extreme cases or 2 per cent on the average, while 3 by 6-in. concrete cylinders dried for a similar period will give long-time shrinkage values reliable within about 11 per cent in extreme cases or 5 per cent on the average.

Due to the wide range of cements tested, it is considered that the long-time shrinkage of concrete containing any portland or portland-puzzolan cement produced commercially at the present time, could be determined with reasonable accuracy from the results of a relatively short-time test modified by a suitable conversion factor determined experimentally beforehand. The value of this factor would depend upon the size and type of specimen, the mix used, the conditions of the short-time test, etc. If a factor be determined for a particular type of cement it appears reasonable that it would be more reliable for any cement of that type than an average factor determined for a wide range of cements.

If results are required within 28 days, it is believed that long-time shrinkage values correct to within about 8 per cent in extreme cases or to within about 3 per cent on the average could be obtained from mortar bars cured for 7 days and dried for 21 days in air of 50 per cent relative humidity at 70°F. as Table 8 shows an average error in prediction of long-time shrinkage of 5 per cent after 14 days drying and an average of 2 per cent after 28 days drying.

Accelerated Tests—It is possible that more rapid and more accurate determinations could be made by air-drying the specimens at a higher temperature than 70°F. or a lower relative humidity than 50 per cent. However, too rapid a rate of drying of the specimen must be avoided as an excessive moisture gradient on any section of the specimen will result in crazing or checking on the surface. Also, the temperature change between curing and drying conditions must not be large or thermal stresses will be introduced, with possible cracking of the specimen; and high curing temperatures may be undesirable due to changes in the nature of hydration under such conditions.

Results of Pencil Tests—The results of the shrinkage tests on all mortar and neat cement pencils for which the mixes were of uniform consistency, for both normal and accelerated drying, are of no value in predicting the long-time shrinkage of concrete made of the same cement. In a previous test program neat-cement pencils also gave unsatisfactory results for water-cement ratios the same as corresponding concretes. The lack of constancy for a variety of cements of the relative short-time shrinkages of cement mortar and neat cement pencils in comparison with the long-time shrinkage of corresponding concrete cylinders is probably interrelated, at least in part, with the same factor which causes lack of agreement between the relative water-cement ratios for concretes of equal consistency and for corresponding cement pastes and mortars of equal consistency.

CONCLUSIONS

The following conclusions may be drawn from the test program:

1. It is possible to determine the *relative* gross long-time shrinkage of concretes containing different cements from a fairly short-time test.
2. The *actual* long-time shrinkage may be computed from the short-time shrinkage provided a proper conversion factor for the special test conditions has been determined previously. (See Table 7 for short-time shrinkage expressed as a percentage of long-time concrete shrinkage).
3. A satisfactory specimen for a short-time test of relative concrete shrinkage appears to be a 1½ by 1½ by 12-in. mortar bar having the same water-cement ratio as that for concrete of the desired consistency. The bar should be cured 7 days and dried 21 days or longer if possible.
4. Short-time tests on neat-cement and mortar pencils of size ½ by 6-in. cannot be used to predict satisfactorily the long-time shrinkage of corresponding concrete.

Discussion, to close in April 1939 JOURNAL, *should reach A. C. I. Secretary in triplicate by Feb. 1.*

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Bond Creep and Shrinkage Effects in Reinforced Concrete*

By J. R. Shank†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

The nature of bond in reinforced concrete is such that it is possible for bars to move with respect to the concrete envelope without breaking the bond. Two series of tests were made on plain round bars in compression lap splices. One indicated that the bond creep is independent of the distance between bars and the other that it may be comparatively high, as much as 0.04 in. when the bond unit stresses are high, and that it is not proportional to the compressive strength of the concrete. Shrinkage stresses observed in the steel of similar but unloaded specimens indicated that bond creep serves to aid in the elimination of practically all of the shrinkage stresses over a cycle of a year or more. It is proposed that bond resistance be studied from the standpoint of friction and therefore from the standpoints of volume change and surface conditions of the steel and structural details rather than from compressive strength.

Measurements of plastic flow shortening of reinforced concrete columns in buildings have indicated at various times rather high stresses in the reinforcing steel. When such measurements are taken over several stories some uncertainty arises because of the bar splices that usually occur at or just above the floors. Whether or not movement of the bars with respect to the surrounding concrete is possible, or may take place without breaking the bond, is a question that enters into the determination of the actual stresses set up in the steel by the plastic flow of the concrete. If the bars may move past each other at these splices only a very small amount, the stresses indicated by the overall measurements must be materially reduced. The effects of shrinkage are also tied up with this possible bond movement or bond creep.

*Received by the Institute Sept. 14, 1938
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As a part of some studies of this question, two series of specimens were made up at The Ohio State University to simulate the compression bar splice and were placed under sustained load for some hundreds of days, during which time measurements were made on the movements of the bars past each other. One series was reported in the Proceedings of the American Concrete Institute of 1931[1].

The question asked in the first series was whether or not movement of the bars past each other is possible and if so what effect would the distance between bars have on it. Four specimens were made identically save for the distance between the bars, which were close, one-half inch, one inch, and one and one-half inches. Loadings were the same for all specimens. The specimen having the one and one-half inches distance between bars broke in shear or diagonal tension through the concrete. Another specimen with bars spaced one and one-half inch was made and tested at half the regular loading and the movements were multiplied by two. The answer was that movement of the bars is possible, and its amount is independent of the distance between bars. This latter observation indicates that the movement is in the nature of slip at the bar surfaces rather than as a result of shear or other possible distortions in the concrete.

In the second series,* the question asked was whether the bond creep has any definite relation to the quality of the concrete as defined by the cement-water ratio or the 28-day standard compressive strength. The answer was that the creeps are different in the different quality specimens but no definite relation could be observed. One outstanding observation was that the bond creep is decidedly erratic. The movment in one specimen was as much as 0.04 in. in 800 days without release of the load.

The specimen for the second series, which is quite similar to that of the first is shown on the upper part of Fig. 1. It will be noted that at one end of the specimen one bar projects beyond the other, and at the other end this condition is reversed. The load applications were made through cast-iron brick rattler balls resting in slight cuppings in the end faces of these longer projecting bar ends. The entire load passed from one bar by bond into the concrete encasement and thence by bond into the other bar. The loading was produced and sustained by means of a gravity loading device in which the specimen occupied the compression side of a gravity loaded beam[2]. The amount of loading was measured and the weight adjusted by the use of the strain-

1. Discussion by J. R. Shank of "Flow of Concrete under the Action of Sustained Loads" by R. E. Davis and H. E. Davis, JOURNAL Amer. Concrete Inst., June 1931; *Proceedings*, Vol. 27 1931, p. 1286

*The author was assisted in this second series by Charles H. Black and H. L. Moss candidates for the degree of Bachelor of Civil Engineering at the Ohio State University.

2. Discussion by J. R. Shank, "Flow of Concrete under Sustained Compressive Stress" by Raymond E. Davis, A. C. I. *Proceedings*, Vol. 24, 1928, p. 327.

gage on a calibrated steel bar in the place of the test specimen. It may be noted that the only possible variation in the constancy of the sustained load must come by way of variations in the weight on the beam loading device, and that variations in temperature or humidity on the specimen can have no effect.

Bar movements were measured by means of a 20-inch fulcrum-plate strain-gage operating in No. 56 drill holes on two sides of both bars. One drill hole was selected as a pivot from which measurements were made to both points at the far end, one on the same bar and the other on the other bar. From such data it was possible to determine the movement of the bar at one far hole past the far hole of the other bar. All holes were used as pivots and the movements were all averaged to produce the movement of one bar past the other.

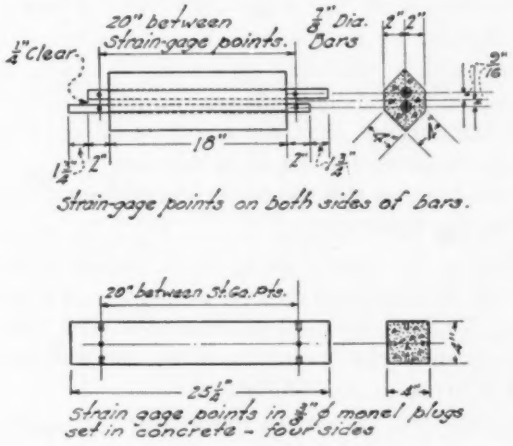

FIG. 1

For the specimens of the second series of tests five different cement-water weight ratios were selected: 1.2, 1.4, 1.6, 1.8, and 2.0 (volumet-

TABLE 1—MATERIALS AND PROPORTIONS DATA

Cement	Proportions by Weight				Slump inches	Ratio by Weight Cement / Total Solids	Ratio by Weight Water / Total Solids	Ratio Cement / Paste Total Volume	28-day Comp. Strength lbs. per sq. in.
Water by wt.	Cement	Water	Sand	Gravel					
1.2	1	0.83	3.33	5.00	5½	0.107	0.89	0.269	2590
1.4	1	0.71	2.60	3.93	5	0.132	0.94	0.296	3480
1.6	1	0.63	2.39	3.50	4¾	0.145	0.90	0.300	4190
1.8	1	0.56	2.04	3.06	4½	0.164	0.91	0.313	4360
2.0	1	0.50	1.75	2.63	4	0.186	0.93	0.332	4880
1.9[1]	1	0.53	2.24	2.52	3	0.174	0.92	0.322	4950

ric water-cement ratios of 1.25, 1.075, 0.94, 0.84 and 0.75 respectively). The proportions are shown in Table 1. The bars used were 7/8 inch round plain hot rolled steel.

After these specimens and their attending compression cylinders were cured at 70 degrees Fahrenheit and 100 per cent humidity for 28 days, the cylinders were tested in compression and one specimen of each mix was loaded in the gravity loading device.

The loads imposed on the specimens, excepting for those of c/w = 1.2 and 1.4, are proportional to the 28-day compressive strengths. The loads intended for the two exceptions were the proportional loads 6430 and 8630 pounds. After these two were loaded, an error was found in the plotting of the calibration curve for the load measuring bar. The actual loads were 5290 and 7600 respectively, as given on Fig. 2. The ordinates of Fig. 2 are the observed creep data for the load imposed divided by the average bond stresses. That the bond creep is proportional to the bond stress has not been demonstrated. However, it was found from three separate sources, R. E. Davis, W. H. Glanville[4], and the author[3], that the plastic flow or creep of concrete is closely proportional to the stress in working ranges. To get the creep for any bond stress, the values for the ordinates of Fig. 2 should be multiplied by the bond stress.

This conforms to the procedure in the discussion of 1931[1]. One of the curves of 1931 is re-plotted on Fig. 2 as c/w = 1.9 for the purpose of comparison. To find the total movement at any time from Fig. 2 it is necessary to multiply the ordinate by the average bond stress applied, which is given in the tabulation.

At the left of Fig. 2 is shown the movements at loading. With the exception of c/w = 1.2 and the old test c/w = 1.9, the movements at loading are seen to be nearly proportional to the bond stress or to the total load applied. The specimen for c/w = 1.4 appears to be slightly weak and that for c/w = 1.2 to be strong. The placing of the concrete in making the specimen c/w = 1.2 was difficult and may therefore have received more working during placing. The movements at loading are a combination of deformations in the steel due to the changes in the stress in the steel, some shear and diagonal tension distortion, and some bond movement at the surface of the bars. After loading, the steel deformation changes were absent, and the 1931 discussion indicated that shear and diagonal tension plastic distortions are relatively small. The time movements shown may thus be said to

3. "The Mechanics of Plastic Flow of Concrete" by J. R. Shank, JOURNAL Amer. Concrete Inst., Nov.-Dec. 1935; *Proceedings*, Vol. 32, p. 165.
4. W. H. Glanville, "Studies in Reinforced Concrete, II—Shrinkage Stresses." Technical Paper No. 11, Dept. of Scientific and Industrial Research, England.

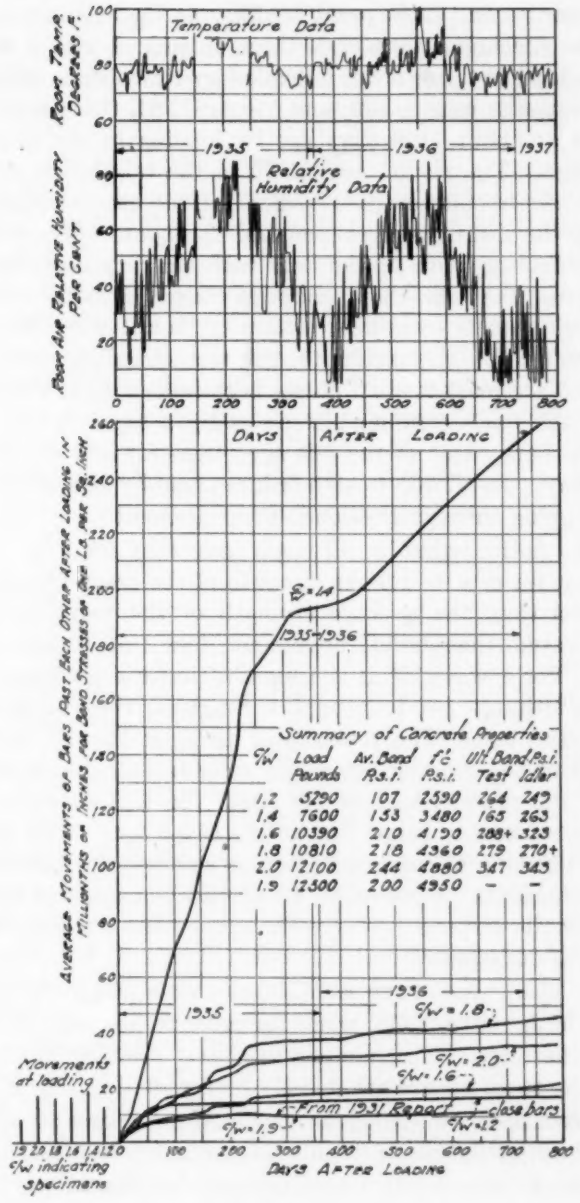
Temperature Data
Room Temp. Degrees F.
Relative Humidity Data
Room Air Relative Humidity Per Cent
1935
1936
1937
Days After Loading
Average Movements on Bars Past Each Other After Loading in Millionths on Inches per Bond Stresses on One Sq. Inch
E=14
1935-1936
Summary of Concrete Properties
c/w | Load Pounds | Av. Bond P.S.I. | f'c P.S.I. | Ult. Bond P.S.I. Test | Ult. Bond P.S.I. Idler
1.2 | 5290 | 107 | 2590 | 264 | 249
1.4 | 7600 | 153 | 3480 | 165 | 263
1.6 | 10390 | 210 | 4190 | 288+ | 323
1.8 | 10810 | 218 | 4360 | 279 | 270+
2.0 | 12100 | 244 | 4880 | 347 | 343
1.9 | 12500 | 200 | 4950 | – | –
c/w = 1.8
c/w = 2.0
c/w = 1.6
c/w = 1.9
c/w = 1.2
From 1931 Report – close bars
Movements at loading
1.9 2.0 1.8 1.6 1.4 1.2 1.0
c/w indicating Specimens
Days After Loading

FIG. 2

be due largely to bond creep which may be thought of as a continuation of small ruptures and subsequent healings at the surfaces of the bars.

When the specimens were cast, three of each of a kind were made, one for the load test, one as an unloaded specimen upon which shrinkage measurements were taken, and a spare. All three were tested to destruction at about 1100 days age by loading in the same manner as in the test. The results are shown in the tabulation at the right of Fig. 2. The results marked "Idler" at the extreme right are the average for the shrinkage specimen and the spare. The slight inclinations of the long ends of the bars with the direction of loading set up bending in the bars that produced elastic limit fiber stresses at about 14,000 pounds total load (282 p.s.i. bond) when the load was perfectly centered. The results for the test specimen c/w = 1.6 and the Idler *1.8* are marked with pluses because failure in each case was caused by the bar's bending instead of by a slipping of one or the other of the bars. One of the idle specimens for c/w = 1.8 failed in the same way. In all other cases failure came by one or the other of the bars slipping through the concrete encasement.

It will be noticed that in all cases save that for c/w = 1.4 and possibly that for c/w = 1.8 the strength of the creep-tested specimen was not less than the average strength of the two idle specimens which indicated that bond movement does not impair the bond resistance. For c/w = 1.4, it is apparent that the specimen was very near to its ultimate bond strength throughout the 795 days that it was under the test. The remarkable thing is that it did not fail. The nature of the loading procedure removed all doubt of the uniformity of the maintenance of the load. In this case the ultimate bond was considerably below that of the two idlers. This test affords a basis for thinking of the bond creep as a series of local failures and subsequent healings. For c/w = 1.4 the healing was not adequate to make up for the failures. Where the movement was not so great it was more than adequate.

All of the specimens of the second series were put under load about the middle of January, 1935, when the relative humidity of the room air was rather low. The temperature and relative humidity conditions of the room air are graphically represented in Fig. 2.

Two differences between the actions in these tests and what might be expected for bar splices in reinforced concrete columns are apparent. (1) Plain bars were used in the test; deformed bars are likely to be used in construction. It is expected that bar deformations would have prevented the large movement of *c/w = 1.4* and possibly have reduced the smaller ones. These test results add a new angle to the question

of the effecacy of deformed bars. (2) In these test specimens there was no concrete at the ends of the spliced bars as there would be in a constructed column. Little or no end resistance to bond creep movement may be expected at the end of the upper bar of a splice in a column, but the end of the lower bar would ordinarily be well covered.

SHRINKAGE EFFECTS

Strain-gage measurements on the unloaded specimen of the second series offered data of some interest on the question of stresses set up in reinforced concrete due to the shrinkage of the concrete. Information concerning the shrinkage of these concretes in the plain form was obtained by taking strain-gage measurements on 4 x 4 inch specimens as shown in Fig. 1. For about 50 days or to the end of February 1935, the data on the plain specimens, when plotted, deformations as ordinates and days as abscissae, took the form $y = Cx^a$, where C is a constant for the concrete. Values for C, a, and limits for x are shown in Table 2. After 50 days or thereabouts these curves flattened out showing very little further shrinkage deformation.

TABLE 2

Curve Formula $y = Cx_1$

c/w	0 to x_1 days			x_1 to x_2 days			x_2 to 550 days	
	x_1	C	a	x_2	C	a	C	a
1.2	4.2	40	0.88	55.0	72	0.47	410	0.035
1.4	41.5	53	0.59	41.5	53	0.59	360	0.079
1.6	10.0	25	0.91	49.5	46	0.56	290	0.089
1.8	16.0	40	0.70	48.0	92	0.41	350	0.060
2.0	11.5	35	0.82	48.0	92	0.42	440	0.013
Av.	16.6	39	0.78	48.4	71	0.49	370	0.058

In order to show what took place in these reinforced concrete specimens, computations were made to determine the total stress P that should be obtained in the steel due to the shrinkage if absolutely no movement of the steel with respect to the concrete took place in the full length of embedment. The shrinkage data were taken from the plain specimens. These values were compared with the total stress values S computed from the strain-gage measurements on the bars. The formula for P was given in an earlier paper[3].

Values for P and S are plotted on Fig. 3. It is apparent that S can never be as great as P for any one time because it is impossible to have absolutely no movement of the steel with respect to the concrete. The difference between the two must be accounted for by slippage at the ends due to local bond failures, bond creep at the ends, and cracking of the concrete due to the tension set up. A picture of what

takes place in specimens like these is given by Glanville[4] and another by C. P. Vetter[5] with discussion by Beyer and Solakian. Data for the former paper were obtained by direct measurements on reinforced concrete; for the latter photo-elastically on a bakelite model rein-

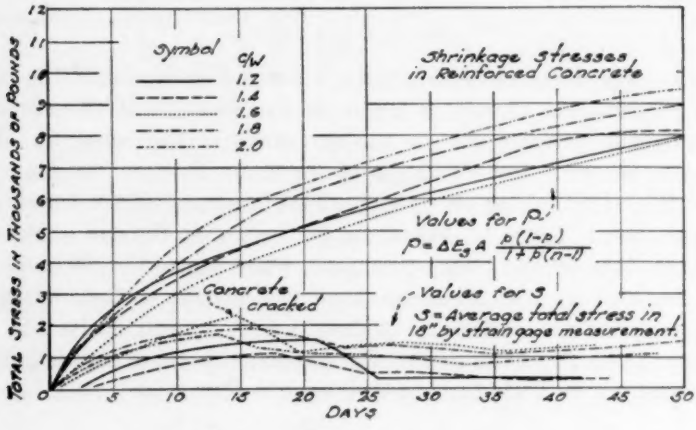

FIG. 3.

$$P = E_s \Delta A \frac{p(1-p)}{1+p(n-1)}$$

P, Total stress in the concrete in tension or the steel in compression, pounds.
Δ, Plain concrete shrinkage in inches per inch.
E_s, Modulus of Elasticity for steel.
A, Gross area of cross-section.
p, Proportion of steel cross-sectional area in the gross cross-section.
n, Elastic modular ratio E_s/E_c.

forced with an aluminum rod. From these two sources it appears that bond stresses occur only at the ends of the rods and cover only such distances in from the ends as are necessary to accumulate stresses equal to the value P. Glanville's work indicated constant bond or uniform stress accumulation in this distance. The other work digressed from this hardly enough to indicate any other rate of accumulation. Between these two sections near the ends of the rods the stresses in the concrete and steel remain constant and there is therefore no stress to cause breaking of the bond. It is apparent that the full value of P cannot obtain at any point on a specimen shorter than twice this necessary end bond distance, and that the longer a specimen is the nearer the average total stress S approaches the value P. When the length of specimen is twice this bond distance, S is just half of P.

5. C. P. Vetter, "Stresses in Reinforced Concrete Due to Volume Changes," Trans. A. S. C. E. Vol. 98, pp. 1039 and 1064.

The effect of bond creep is to lengthen the required bond distance and so cause a decrease in the ratio S/P.

In this series of tests bond creep was evidenced in the specimens for c/w ratios of 2.0, 1.8, and 1.6 by a falling off of the ratio S/P up to the time when cracks apparently occurred in the concrete. See Fig. 3. The ratio S/P for c/w = 1.6 started at 0.65 and declined steadily until at 15 days it was just 0.50. In the cases of the c/w ratios of 1.4 and 1.2, the S/P ratio started and remained at about 0.20.

After the first 50 days the shrinkage stresses continued to reduce, reaching zero in from 85 to 145 days, or during the spring or early summer. Reversals of stress followed after the times of zero stress, reaching maximums at from 170 to 210 days. The specimen for c/w equals 1.6 reached a reversal of 1170 pounds total stress at 175 days. The average amount was 394 pounds at an average time of 152 days. The specimen for c/w equals 1.4 showed no reversal. In from 150 to 430 days, shrinkage stresses again occurred averaging 675 pounds total stress.

The results of these tests indicate that the high stresses in the steel which are indicated by plastic flow calculations are reduced considerably by the possibility of bond creep, particularly at joints, knees, and similar complex situations where a large proportion of the steel bars are ended either at far sides or in splices. In line with the reduction due to plastic flow of concrete stresses we now find a reduction of steel stresses because of the bond creep. This is a happy situation since it is the nature of reinforced concrete to be constructed with continuous or fixed end members and complex joint conditions. The designer may want to take this bond creep possibility into account when designating hooks, or straight bar endings when detailing complex and important member connections. Individual member design is simple compared with that of designing proper details, and the details are no less important.

These tests indicate that shrinkage stresses are practically eliminated after a full cycle of humidity changes have acted. A large part of this is due to the cracking of the concrete, but the bond creeps in the vicinities of the cracks, as well as along the entire contact area, are necessary to bring the stresses to a balance over a humidity cycle.

A series of tests to find out the effect of bar deformations on bond creep is suggested. A new light may be thrown on the efficacy of deformed bars, as it is generally thought that bar deformations come into play only after some slight initial slippage has taken place.

These tests, together with others[6] indicate that bond is not proportional directly to compressive strength.

Indications are strong that bond resistance is developed more because of friction than because of adhesion. Adhesion certainly occurs between cement and aggregate, and adhesion may be expected between cement and rock or other earthy surfaces, including glasses, but to metallic surfaces it is questioned. Friction must come about by the shrinkage action of the concrete. Shrinkage is produced by humidity and temperature effects and the friction set up by it on the tensile value of the concrete. This all suggests that tests for bond strength should be carried on parallel with tests for shrinkage and other volume change conditions, and tensile strength rather than with compressive strength.

6. F. G. Thomas, "Cracking in Reinforced Concrete" *Structural Engineer*, England, 1936, 14 (new series) 7 pp. 298-320.

Discussion, to close in April 1939* JOURNAL, *should reach A. C. I. Secretary in triplicate by Feb. 1.

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A Precise Moment Distribution Method*

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SYNOPSIS

A modification of the Moment Distribution Method is herein presented in which "precise" carry-over and distribution coefficients are first computed for the frame. Thereafter the fundamental principles of the Moment Distribution Method are followed. Repeated cycles of carry-over and distribution are thereby avoided. An additional short-cut is based on the use of restrained end moments derived from the fixed end moments. For beams and frames with four or more members, particularly where the effect of various loading conditions is to be studied, it is believed that this method will be found rapid and accurate.

Diagrams for various loading conditions and for beams of varying moment of inertia are included. The concepts underlying this method are not new, having been presented by T. Y. Lin, Proc. A.S.C.E., Vol. 60, 1934, p. 1451, "A Direct Method of Moment Distribution," and also by Suter, "Die Methode der Festpunkte," and Strassner, "Neuere Methoden." The particular technique developed here, however, is new and it is believed to be more rapid and simpler than the previous procedures.

The method of analysis herein presented is a modification of the Moment Distribution Method, in which precise carry-over and distribution coefficients are used. In the usual Moment Distribution Method the moment transmitted to the far end of a member is ½ of the moment at the near end and this coefficient, ½, is based upon the assumption that the far end of the member is temporarily fixed against rotation. If we take into consideration the flexural properties of the members which enter the joint at the far end, it is possible to obtain a coefficient α, in place of the value ½. This precise carry-over coefficient α will determine the proper amount of moment to be carried over, giving due consideration to the restraint at the far end of the member. Its value, ordinarily, will be between 0 and .5.

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When the far end of the beam is simply supported, α will be 0 and when it is fixed α will be .5. In distributing an unbalanced moment at a joint, the distribution is made in proportion to $\varkappa$ instead of in proportion to K. $\varkappa$ will be termed the precise distribution factor. In the Appendix it will be shown that

$$\varkappa = \frac{K}{2 - \alpha} \quad \ldots\ldots (1)$$

In each span there will be two carry-over coefficients corresponding respectively to a carry-over from left to right and to a carry-over from right to left. Likewise, there will be two distribution coefficients determined from their respective α's. K, as usual, is $\frac{I}{L}$. The precise distribution coefficient $\varkappa$ at one end of the member depends upon K and upon the carry-over coefficient at that end. But this carry-over coefficient at the one end in turn will depend upon the conditions and restraints at that end. Consequently, $\varkappa$ will be a measure of the distribution of moments at the other end of the beam. This will also be proved in the Appendix.

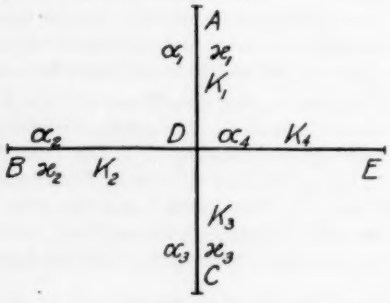


FIG. 1

The carry-over coefficients at joints with partial restraint are determined as indicated in Fig. 1 by eq. (2).

$$\alpha_4 = \frac{\varkappa_1 + \varkappa_2 + \varkappa_3}{K_4 + 2(\varkappa_1 + \varkappa_2 + \varkappa_3)} \quad \ldots\ldots (2)$$

This result is also derived in the Appendix. In some cases it will be necessary to obtain an approximate value of the carry-over coefficient because the distribution coefficients preceding it are not all determined. This can be obtained by means of eq. (3).

$$\text{Approx. } \alpha_4 = \frac{2(K_1 + K_2 + K_3)}{3K_4 + 4(K_1 + K_2 + K_3)} \quad \ldots\ldots (3)$$

Note that the carry-over coefficient, α_4, at the near end of the member DE is determined by the distribution coefficients at the far ends of

the members BD, AD, CD which enter this nearest joint D. In obtaining these precise carry-over coefficients we start from points in the structure at which the conditions of restraint are known. At every point where we have freely supported members we write down the carry-over coefficients $\alpha = 0$ and wherever we have fixed ends we indicate the carry-over coefficients as $\alpha = .5$. We then determine the distribution coefficients ($\varkappa$'s) corresponding to these α's. From these known values of $\varkappa$ we then proceed to the next bay and obtain the value of the carry-over coefficient α by applying eq. (2). If we have an insufficient number of $\varkappa$-values we first determine this α approximately by means of eq. (3). We proceed throughout the whole frame in this manner and we will find that we will finally be able to correct all of the approximate α-values and the corresponding $\varkappa$-values and thus obtain the precise carry-over and distribution coefficients. The change from the approximate to the precise value of these coefficients will be relatively small and will not appreciably influence the value of the next succeeding coefficients.

Having obtained the precise carry-over and distribution coefficients we use them in the same manner as the usual approximate coefficients and distribute and carry-over the unbalanced moment throughout the frame. In this Precise Moment Distribution Method, however, it will be found that no successive approximations are required, because a moment carried over to the far end of the beam is not redistributed so as to retrace its path. Thus one set of carry-overs and distributions throughout the frame for each unbalanced moment is all that is required and furthermore, any desired degree of accuracy can be obtained in this one step. It will be found that the formulae for the precise carry-over and distribution coefficients are very easily remembered and that the computations can be carried through on a slide rule with comparative ease. A further development is presented in which restrained end moments are computed directly from the fixed end moments and the carry-over coefficients. Thus it is possible to determine the moments at the ends of a beam in a rigid frame directly for a load on the beam without carrying through the process of moment distribution. Of course, if it is desired to find the effect of the load in one span upon the moments in other spans, then it will be necessary to proceed with the distribution and carry-over of unbalanced moments. In this case the restrained end moments can be used as a point of departure in place of fixed end moments and in a similar manner with the advantage that two sets of carry-overs and distributions are avoided. Hereafter, fixed end moments will be abbreviated F. E. M. and restrained end moments will be abbreviated R. E. M.

Finally, the method is extended to haunched beams and beams of varying cross-section by means of curves from which certain coefficients can be obtained. The use of all these principles is illustrated by a number of typical problems.

The Slope-Deflection convention of signs is employed, because it appears simpler and easier to apply. Moments will be considered as applied externally to the ends of the beams or columns when these are cut close to the joints, and clockwise moments thus applied will be considered positive. The convention of signs is not material to the method as any consistent convention can be used.

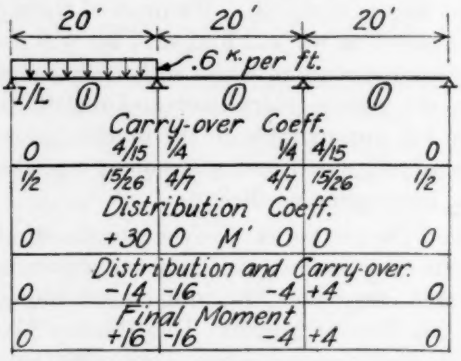


FIG. 2

APPLICATION TO CONTINUOUS BEAM

Steps

1.) Draw sketch of beam and loading and compute $\frac{I}{L}$ for each span.

2.) Compute carry-over and distribution coefficients, using eq. (2) for α and eq. (1) for $\varkappa$.

a.) At A, freely supported end, $\alpha = 0$. $\varkappa = \frac{1}{2 - 0} = \frac{1}{2}$.

b.) At left end of span BC, $\alpha = \frac{\frac{1}{2}}{1 + 2(\frac{1}{2})} = \frac{1}{4}$. (eq. (1).

$$\varkappa = \frac{1}{2 - \frac{1}{4}} = 4/7$$

c.) At left end of span CD, $\alpha = \frac{4/7}{1 + 2(4/7)} = 4/15$.

$$\varkappa = \frac{1}{2 - 4/15} = 15/26.$$

d.) Beginning at D and proceeding successively to spans to left. $\alpha = 0$, $\varkappa = \frac{1}{2}$, $\alpha = \frac{1}{4}$, $\varkappa = 4/7$, $\alpha = 4/15$, $\varkappa = 15/26$.

3.) Write down M', the fixed beam moments. In first span, with left end simply supported $M' = \frac{wL^2}{8}$ (same as in ordinary moment distribution method.) It is positive in accordance with convention of signs. No other fixed beam moments occur in this case.

4.) *a.*) Distribute unbalanced moments. At B unbalanced moment is $+30.0$. Distribution to BC is $\frac{4/7}{1/2 + 4/7} = \frac{8}{15}$, and to BA it is $\frac{1/2}{1/2 + 4/7} = \frac{7}{15}$, of unbalanced moment, or -16 and -14 respectively. Note that distribution is made using distribution factors at ends of beams farthest from joint about which distribution is being made. Also the signs of the distributed moments are opposite those of the unbalanced moments, since we are, in effect, introducing a moment equal to and opposite in sign to the unbalanced moment at B to balance the moments.

b.) Carry over the distributed moments. The carry-over from B to C is $\frac{1}{4} \times -16 = -4$. We use the right carry-over factor in Span BC.

c.) Distribute the carried over moment. In this case, the moment -4 at C in BC merely gives rise to a moment $+4$ at C in CD. Note that the moment -4 at C in BC is now regarded as an external moment applied to remainder of structure.

d.) Carry over and distribute as above until moments become insignificantly small. In this case the process ends at D, where the carried over moment is 0.

5.) Add all the moments algebraically, to get the actual moments.

APPLICATION TO RIGID FRAMES

The procedure for rigid frames is practically the same as for continuous beams. A one-story bent will be analyzed.

If we have a dead load of 1200 lb. per ft. and a live load of 600 lb. per ft., we get the maximum moment at B in BA, for example, as follows: We note that M_{CD} with load in first span equals M_{BA} with load in third span, with sign reversed.

		M_{BA}
Load of 1^k per ft. in 1st. span		$+57.8$
do. in 2nd. span		$+35.5$
do. in 3rd. span		$-\ 6.8$
For D.L. (1 + 2 + 3)	$+84.4 \times 1.2$	$= +101.3$
For L.L. (1 + 2)	$+91.3 \times .6$	$= +\ 54.8$
M_{BA}, combined D.L. + L.L.		$= +156.1$

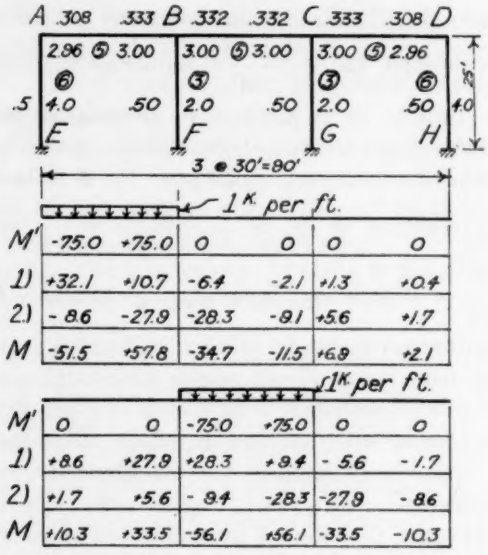
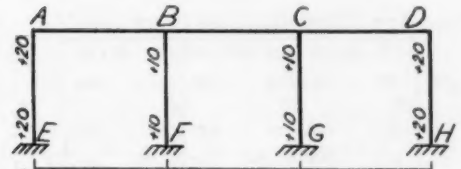


FIG. 3

The carry-over and distribution coefficients can be rapidly calculated by means of a slide rule, as the formulae are easily remembered. At E, F, G, and H, $\alpha = .5$ since the members are fixed. $\varkappa$ at E and $H = \dfrac{6}{2-.5} = 4.0$ and at F and G, $\varkappa = \dfrac{3}{2-.5} = 2.0$ α at A in AB and at D in CD, $= \dfrac{4}{5+2\times 4} = \dfrac{4}{13} = .308$. $\varkappa = \dfrac{5}{2-.308} = 2.96$. α at B and C in BC, $= \dfrac{2+2.96}{5+2(2+2.96)} = .332$ and $\varkappa = \dfrac{5}{2-.332} = 3.00$. α at B in BA and at C in CD, $= \dfrac{2+3.00}{5+2(2+3.00)} = .333$ and $\varkappa = \dfrac{5}{2-.333} = 3.00$. The α and $\varkappa$ values at tops of columns are unnecessary and can be omitted.

The sidesway effect can be included in the same manner as in the ordinary moment distribution method. Thus, in the frame of Fig. 3, for the first loading case, we will have sidesway. We start out with assumed moments proportional to $\dfrac{K}{L}$ of the columns. (Since the columns are all of the same length, we make out assumed moments proportional to K).



M'	0	0	0	0	0	0
1)	-8.6	-2.9	+1.7	+0.6	-0.4	-0.1
2)	-1.1	-3.7	-3.8	-1.3	+0.8	+0.2
3)	-0.2	+0.8	-1.3	-3.8	-3.7	-1.1
4)	-0.1	-0.4	+0.6	+1.7	-2.9	-8.6
M	-9.6	-6.2	-2.8	-2.8	-6.2	-9.6

FIG. 4

Note that we could omit the figures for columns CG and DH, using the symmetry of the frame to get the necessary values. The moment at top of column AE will be $+9.6$ and at the base of column AE, it will be $+20 - \frac{1}{2} \times (9.6 - 20) + 14.8$. $M_{BF} = -(-6.2 - 2.8) = +9.0$, $M_{FB} = +10 + \frac{1}{2}(9.0 - 10) = +9.5$ (since we have subtracted 1.0 ft. kips at top of column and $\frac{1}{2}$ of this *decrement* is carried to base.) The sum of moments at tops and bottoms of columns is $2(9.6 + 14.8) + 2(9.0 + 9.5) = +85.8$. In the original problem the moments were: $M_{AE} = +51.5$, $M_{EA} = +25.8$, $M_{BF} = -(57.8 - 34.7) = -23.1$, $M_{FB} = -11.6$, $M_{CG} = +4.6$, $M_{GC} = +2.3$, $M_{DH} = -2.1$, $M_{HD} = -1.0$. Sum of moments, tops and bottoms of columns $= +46.4$. Therefore our moments in original solution must be changed by the amount of $\frac{46.4}{85.8}$ times moments in assumed side-sway solution. Hence $M_{AB} = -51.5 - 9.6 \times \frac{46.4}{85.8} = -46.3$. $M_{BA} = +57.8 + 6.2 \times \frac{46.4}{85.8} = +61.2$. $M_{BC} = -34.7 - 2.8 \times \frac{46.4}{85.8} = -33.2$. $M_{CB} = -11.5 + 1.5 \times \frac{46.4}{85.8} = -10.0$. $M_{CD} = +6.9 + 3.4 = 10.3$. $M_{DC} = + 2.1 + 5.2 = +7.3$.

In a multi-storied frame one new difficulty is encountered.

We start with $\alpha = .5$ at P, Q, R, and S. $\varkappa$ will then be $\frac{4}{2 - .5} = 2.67$ at P and S and $\frac{3}{2 - .5} = 2.0$ at Q and R. We then find it impossible to go further with our process, since there is no joint at which all $\varkappa$'s except one are known. We therefore use the approximate eq. (3)

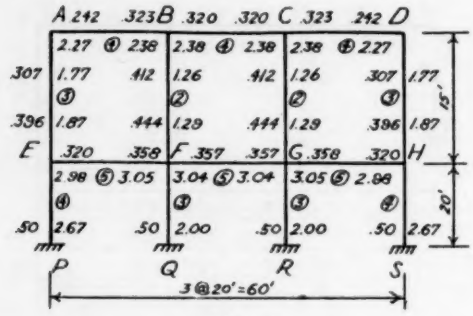


FIG. 5

and apply it at A and D to find α_{AE} and α_{DA}. (α near A in AE, etc.)

$\alpha_{AE} = \dfrac{2 \times 4}{3 \times 3 + 4 \times 4} = .320 = \alpha_{DH}$ and $\varkappa_{AE} = \varkappa_{DH} = \dfrac{3}{2 - .320} = 1.78$.

We also approximate to α_{BF} and α_{CG} in the same way. $\alpha_{BF} = \dfrac{2\,(4 + 4)}{3 \times 2 + 4\,(4 + 4)} = \dfrac{16}{38} = .421 = \alpha_{CG}$ and $\alpha_{BF} = \alpha_{CG} = \dfrac{2}{2 - .421} = 1.27$. We can then obtain, in succession, α_{EF}, α_{HG}, α_{FG}, α_{GF}, α_{FE}, α_{GH}, α_{EA}, α_{FB}, α_{GC}, α_{HD}, α_{AB}, α_{DC}, α_{BC}, α_{CB}, α_{CD}, α_{BA}, and the corresponding $\varkappa$'s.

We next correct α_{AE} and α_{DH} from the now known value of $\varkappa_{BA}$. This gives us $\alpha_{AE} = \alpha_{DH} = .307$ and $\varkappa_{AE} = \varkappa_{DH} = 1.77$. These values replace the original approximations. The change in $\varkappa_{AE}$ is so small that no further correction need be made in α_{EF}. If a larger change had occurred in $\varkappa_{AE}$ we would have also corrected α_{EF}. Similarly, α_{BF}, α_{CG}, $\varkappa_{BF}$, and $\varkappa_{CG}$ are also corrected, and again, the change in $\varkappa$ is too small to have any effect on the remaining $\varkappa$'s.

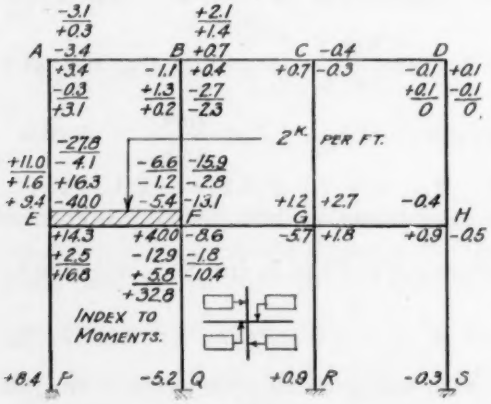


FIG. 6

Fig. 6 shows the moment distributions. The unbalanced moment at E is first distributed, then the unbalanced moment at F. Next we carry over the moment from E to A then to B. Then we carry over the moment from F to B to A. Then we carry over the moments from B to C, F to G, G to C, G to H, H to D, C to D and balance our moments at C. Finally we carry over to moments to P, Q, R, and S.

SOLUTION USING R.E.M.

The restrained end moments (REM) can be obtained from the fixed end moments (FEM) as follows:

FIG. 7

For unsymmetrical loads,

$$\left.\begin{aligned} REM_{AB} &= \left[FEM_{AB}\frac{\alpha(2-\alpha')}{1-\alpha\alpha'} - FEM_{BA}\frac{\alpha(1-2\alpha')}{1-\alpha\alpha'} \right] \\ REM_{BA} &= \left[FEM_{BA}\frac{\alpha'(2-\alpha)}{1-\alpha\alpha'} - FEM_{AB}\frac{\alpha'(1-2\alpha)}{1-\alpha\alpha'} \right] \end{aligned}\right\} \quad \ldots\ldots (4)$$

For symmetrical loads,

$$\left.\begin{aligned} REM_{AB} &= FEM_{AB}\frac{3\alpha(1-\alpha')}{1-\alpha\alpha'} \\ REM_{BA} &= FEM_{BA}\frac{3\alpha'(1-\alpha)}{1-\alpha\alpha'} \end{aligned}\right\} \quad \ldots\ldots (5)$$

as will be shown in the Appendix. These will now be applied to the beam shown in Fig. 2, and the procedure is shown in Fig. 8.

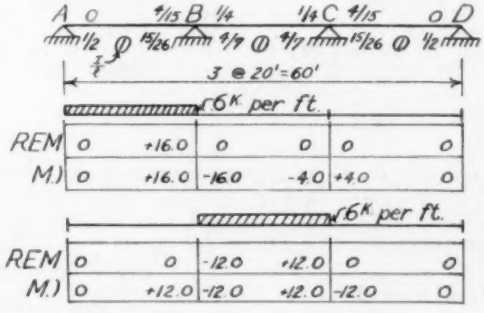


FIG. 8

For load in span AB, $R.E.M.$ at $A = 0$, and at B, it equals $+\frac{wL^2}{12}$.

$\frac{3 \cdot 4/15\ (1-0)}{1-4/15\times 0} = +\frac{0.6\times 400}{15} = +16.0$. In the second step we distribute this to BC, carry over to C and distribute to CD. Note that in distributing moment at B, no moment is redistributed to BA, as this is automatically provided for in the $R.E.M.$ For load in span BC, the $R.E.M.$ at B and at C are equal numerically and are $\frac{0.6\times 400}{12} \times \frac{3\times \frac{1}{4}\times \frac{3}{4}}{1-\frac{1}{4}\cdot\frac{1}{4}} = 12.0$. Again we distribute moments at B and at C. The last line in each case gives the actual moments.

A typical solution involving concentrated loads is shown in Fig. 9.

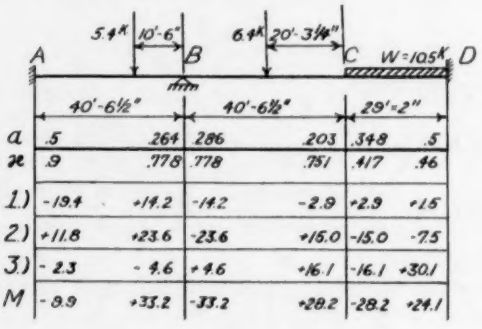


FIG. 9

Step 1. Load in span 1 only, $R.E.M.$ at A, apply eq. (4) $= -\left[10.9\times\frac{.5\ (2-.264)}{1-.5\times.264}+31.1\times\frac{.5\ (1-.528)}{1-.5\times.264}\right] = -19.4$

$R.E.M.$ at B (eq. 5) $= +\left[31.1\times\frac{.264\ (1-.5)}{.868}+10.9\times\frac{.264\ (1-1)}{.868}\right] = +14.2$

Distribute moment at B, $+14.2$ produces moment -14.2 at B in BC.
Carry over to C: $-14.2\times.203 = -2.9$
Distribute, then carry over to D; $+2.9\times.5 = +1.5$.

Step 2. $R.E.M.$ at B in BC, $= -32.5\times\frac{3\times.286\ (1-.203)}{1-.286\times.203} = -23.6$,

$R.E.M.$ at C, $= +32.5\times\frac{3\times.203\ (1-.286)}{1-.286\times.203} = +15.0$ (eq. 5).

Distribute and carry over as before.

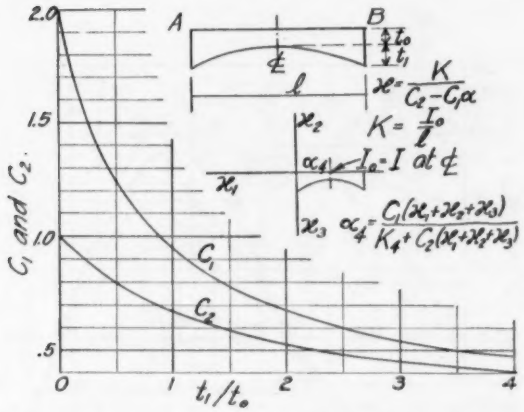


FIG. 10—CARRY-OVER AND DISTRIBUTION COEFFICIENTS, PARABOLIC HAUNCHED BEAMS (SEE ERRATA, P. 109)

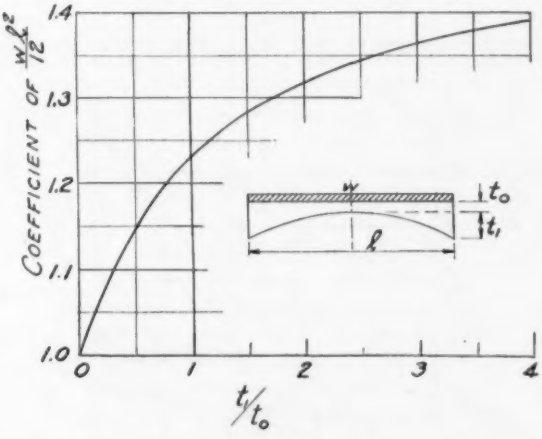


FIG. 11—FIXED END MOMENTS. PARABOLIC HAUNCHED BEAMS UNIFORMLY LOADED

HAUNCHED BEAMS

The only change in the treatment of haunched beams, is in the modification of formulae 1 and 2 and in the fixed end moments. Formula (1) becomes:

$$\varkappa = \frac{K}{C_2 - C_1\alpha} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

and formula (2) becomes,

$$\alpha_4 = \frac{C_1 (\varkappa_1 + \varkappa_2 + \varkappa_3)}{K_4 + C_2 (\varkappa_1 + \varkappa_2 + \varkappa_3)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

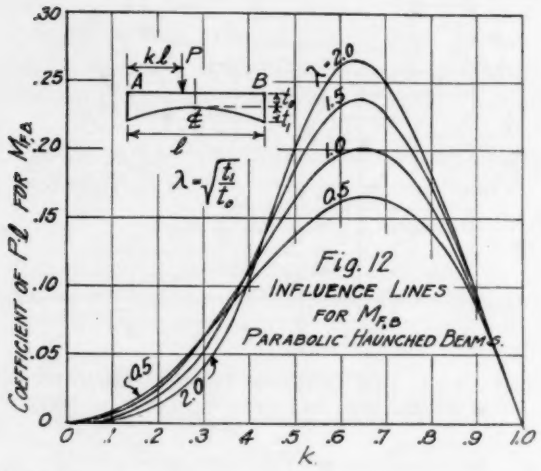


FIG. 12—INFLUENCE LINES FOR $M_{F,B}$, PARABOLIC HAUNCHED BEAMS

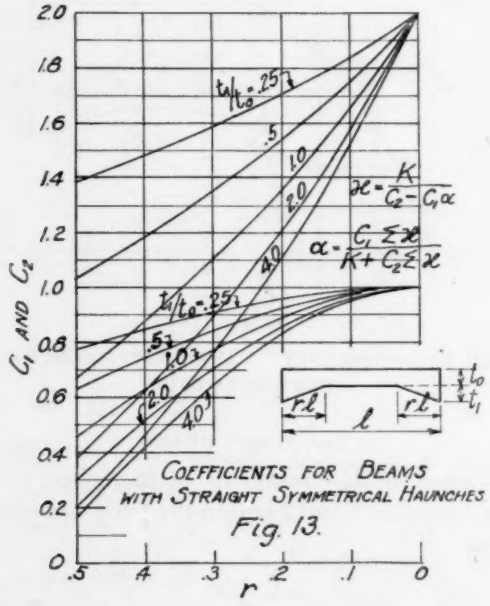


FIG. 13—COEFFICIENTS FOR BEAMS WITH STRAIGHT SYMMETRICAL HAUNCHES (SEE ERRATA, P. 109)

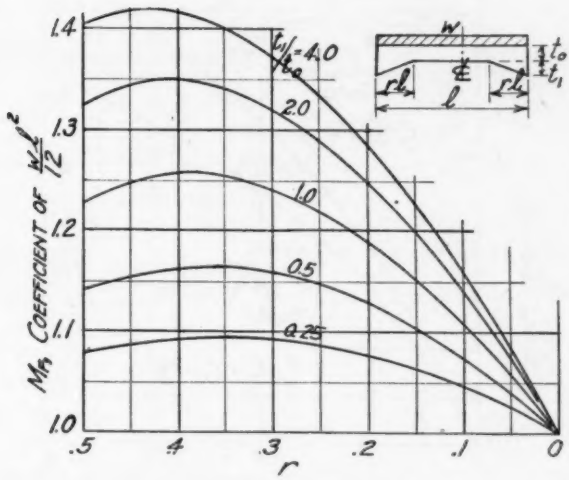


FIG. 14—FIXED END MOMENTS, UNIFORMLY LOADED BEAMS WITH STRAIGHT SYMMETRICAL HAUNCHES

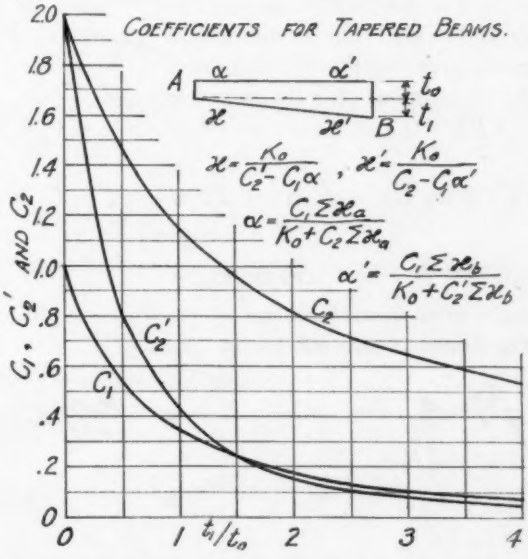


FIG. 15—COEFFICIENTS C_1, C_1', AND C_2 FOR α AND α. STRAIGHT TAPERED BEAMS

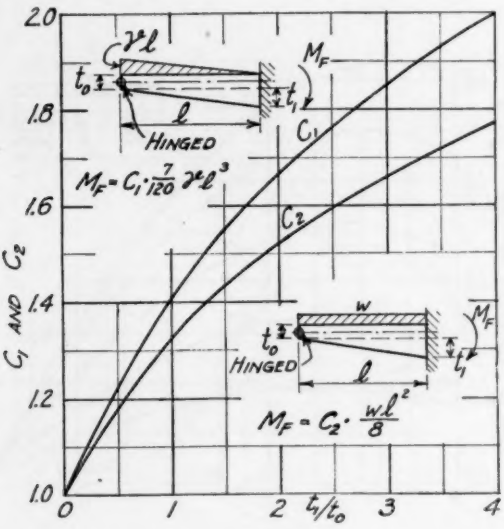


FIG. 16—COEFFICIENTS FOR TAPERED BEAMS, UNIFORM AND UNIFORMLY VARYING LOADS

where C_1 and C_2 are taken from the curves of Figs. 15 and 18. For straight tapered beams, the coefficients are given in Fig. 15. The remaining figures give the fixed end moments.

Restrained end moments for haunched beams.

$$REM_A = \frac{\alpha \left[FEM_A \left(C_2 - C_1 \alpha' \right) + FEM_B \left(C_1 - C_2 \alpha' \right) \right]}{C_1 \left(1 - \alpha\alpha' \right)}$$

$$REM_B = \frac{\alpha' \left[FEM_B \left(C_2 - C_1 \alpha \right) + FEM_A \left(C_1 - C_2 \alpha' \right) \right]}{C_1 \left(1 - \alpha\alpha' \right)}$$

APPENDIX

1.) Derivation of fundamental formulae.

$\varkappa$ = The Precise Distribution Factor.

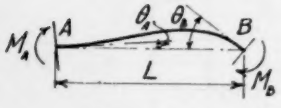


FIG. 17

FIG. 18

Fig. 17 shows a beam of constant section, subjected to a moment M_B, and partly restrained at A, thus producing a moment M_A at A. From the definition of the precise carry-over coefficient α,

$$M_A = \alpha M_B \quad \ldots\ldots (8)$$

Fig. 18 shows the moment diagram.

From the Moment-Area Theorem,

$$\Theta_B = \frac{\frac{1}{2} M_B \cdot L \cdot \frac{2}{3} L - \frac{1}{2} \alpha M_B \cdot L \cdot \frac{1}{3} L}{EI \cdot L}$$

$$M_B = 6E\Theta_B \cdot \frac{K}{2 - \alpha} \quad \ldots\ldots (9)$$

where $K = \frac{I}{L}$.

If we have several beams entering a common joint, as in Fig. 19.

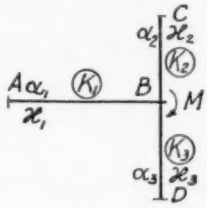


FIG. 19

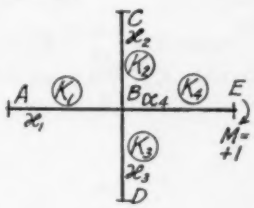


FIG. 20

and a moment M is applied to the joint, then

$$M_{BA} = 6E\Theta_B \cdot \frac{K_1}{2 - \alpha_1} = 6E\Theta_B \cdot \chi_1 \quad \ldots\ldots (10)$$

$$M_{BC} = 6E\Theta_B \cdot \frac{K_2}{2 - \alpha_2} = 6E\Theta_B \cdot \chi_2$$

$$M_{BD} = 6E\Theta_B \cdot \frac{K_3}{2 - \alpha_3} = 6E\Theta_B \cdot \chi_3$$

$$M = M_{BA} + M_{BC} + M_{BD} = 6E\Theta_B (\chi_1 + \chi_2 + \chi_3)$$

$$6E\Theta_B = \frac{M}{\chi_1 + \chi_1 + \chi_3} \quad \ldots\ldots (11)$$

$$M_{BA} = M \cdot \frac{\chi_1}{\chi_1 + \chi_2 + \chi_3} \quad \ldots\ldots (12)$$

and similarly for the other moments. Thus we see that χ is the precise distribution factor, or

$$\chi = \frac{K}{2 - \alpha} \quad \ldots\ldots (1)$$

α = The Precise Carry-over Coefficient.

If we introduce a unit moment at E, then $M_{BE} = +\alpha_4$. The moment diagram will be shown in Fig. 21.

FIG. 21

Then, from the Moment-Area Theorem,

$$6E\Theta_B = \frac{1 - 2\alpha_4}{K_4} \quad \ldots\ldots (13)$$

But, by eq. (11),

$$6E\Theta_B = -\frac{\alpha_4}{\varkappa_1 + \varkappa_2 + \varkappa_3} \quad \ldots\ldots (14)$$

Hence

$$\frac{1 - 2\alpha_4}{K_4} = \frac{\alpha_4}{\varkappa_1 + \varkappa_2 + \varkappa_3}$$

or,

$$\alpha_4 = \frac{\varkappa_1 + \varkappa_2 + \varkappa_3}{K_4 + 2(\varkappa_1 + \varkappa_2 + \varkappa_3)} \quad \ldots\ldots (2)$$

If the ends A, B and C are fixed,

$$\varkappa_1 = \tfrac{2}{3}K_1. \quad \varkappa_2 = \tfrac{2}{3}K_2, \quad \varkappa_3 = \tfrac{2}{3}K_3$$

and

$$\alpha_4 = \frac{2(K_1 + K_2 + K_3)}{3K_4 + 4(K_1 + K_2 + K_3)} \quad \ldots\ldots (3)$$

R.E.M.

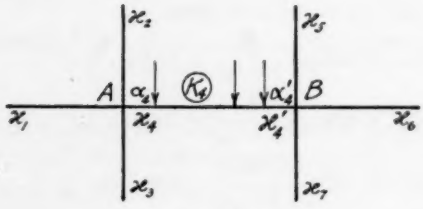


FIG. 22

Distribute FEM_{AB} among members entering joint A. The moment by which FEM_{AB} is reduced is $\frac{\varkappa'_4}{\varkappa'_4 + \varkappa_1 + \varkappa_1 + \varkappa_3}$. Likewise the moment FEM_{BA} is reduced by $\frac{\varkappa_4}{\varkappa'_4 + \varkappa_5 + \varkappa_6 + \varkappa_7}$, when distribution is made at joint B. α_4 times this last moment is carried over to A, so that

$$REM_{AB} = FEM_{AB} - \frac{\varkappa'_4}{\varkappa'_4 + \varkappa_1 + \varkappa_2 + \varkappa_3} FEM_{AB} - \frac{\alpha_4\varkappa_4}{\varkappa_4 + \varkappa_5 + \varkappa_6 + \varkappa_7} FEM_{BA}$$

$$REM_{AB} = FEM_{AB} \frac{\varkappa_1 + \varkappa_2 + \varkappa_3}{\varkappa'_4 + \varkappa_1 + \varkappa_2 + \varkappa_3} - \frac{\alpha_4\varkappa_4}{\varkappa_4 + \varkappa_5 + \varkappa_6 + \varkappa_7} FEM_{BA}$$

But

$$\alpha_4 = \frac{\varkappa_1 + \varkappa_2 + \varkappa_3}{K_4 + 2(\varkappa_1 + \varkappa_2 + \varkappa_3)}$$

$$\alpha'_4 = \frac{\varkappa_5 + \varkappa_6 + \varkappa_7}{K_4 + 2(\varkappa_5 + \varkappa_6 + \varkappa_7)}$$

Therefore,

$$x_1 + x_2 + x_3 = \frac{\alpha_4 K_4}{1 - 2\alpha_4}$$

$$x_5 + x_6 + x_7 = \frac{\alpha'_4 K_4}{1 - 2\alpha'_4}$$

Also $x_4 = \dfrac{K_4}{2 - \alpha_4}$ and $x'_4 = \dfrac{K_4}{2 - \alpha'_4}$

Therefore,

$$REM_{AB} = FEM_{AB} \frac{\dfrac{\alpha_4 K_4}{1 - 2\alpha_4}}{\dfrac{K_4}{2 - \alpha'_4} + \dfrac{\alpha_3 K_3}{1 - 2\alpha_4}} - \frac{\dfrac{\alpha_4 K_4}{2 - \alpha_4}}{\dfrac{K_4}{2 - \alpha_4} + \dfrac{\alpha'_4 K_4}{1 - 2\alpha'_4}} FEM_{BA}$$

or

$$\left.\begin{aligned} REM_{AB} &= FEM_{AB} \frac{\alpha_4(2 - \alpha'_4)}{1 - \alpha'_4\alpha_4} - FEM_{AB} \frac{\alpha_4(1 - 2\alpha'_4)}{1 - \alpha'_4\alpha_4} \\ \text{and similarly} \\ REM_{BA} &= FEM_{BA} \frac{\alpha'_4(2 - \alpha_4)}{1 - \alpha_4\alpha_{14}} - FEM_{AB} \frac{\alpha'^1_4(1 - 2\alpha_4)}{1 - \alpha_4\alpha'_4} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

or symmetrical loads, $FEM_{AB} = -FEM_{BA}$

$$\left.\begin{aligned} REM_{AB} &= FEM_{AB} \frac{3\alpha_4(1 - \alpha'_4)}{1 - \alpha_4\alpha'_4} \\ REM_{BA} &= FEM_{BA} \frac{3\alpha'_4(1 - \alpha_4)}{1 - \alpha_4\alpha'_4} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Discussion, to close in April 1939* JOURNAL, *should reach A. C. I Secretary in triplicate by Feb. 1.

ERRATA—As this paper was on the press and diagrams had passed the stage of correction these errors were noted by the author:

Fig. 10, p. 103: the subscripts 1 and 2 in C_1 and C_2 on the two curves should be interchanged.

Fig. 13, p. 104: Curves converging upper right should be designated C_2; the lower group of curves, C_1.

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AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN *APRIL 1939*

Discussion of a Paper by Joseph A. Wise

A Precise Moment Distribution Method*

BY U. T. BERG, EDWARD L. AGRAMONTE AND CLOSURE BY THE AUTHOR

BY U. T. BERG†

Frame design of the single cycle variety requires rapid solutions for the distribution of moments on a loaded member and for their transmission to other members. The moment of a joint is distributed in direct ratio to the precise distribution factor, $\varkappa$, for which Professor Wise has presented a beautifully simple solution. He obtains the reduced end moments (*REM*) for the loaded span from the fixed end moments (*FEM*). *FEM* values became popular with the slope deflection method. They are essential to Professor Cross' moment distribution method. This writer questions their utility in connection with single cycle moment distributions. He considers the following solution for *REM* as more convenient. Make $M = -\frac{6KE}{C_1}\phi$, where ϕ is the slope at the end considered for load on simple supports.

(These values may also be derived from the *FEM* values thus:

$$M_{AB} = \frac{C_2}{C_1} FEM_{AB} - FEM_{BA};\ M_{BA} = \frac{C'_2}{C_1} FEM_{BA} - FEM_{AB})$$

The solution for a loaded member of constant or variable moments of inertia may be obtained thus:

$$REM_{AB} = \frac{\alpha}{1 - \alpha \cdot \alpha'}[M_{AB} + \alpha' \cdot M_{BA}];$$

$$REM_{BA} = \frac{\alpha'}{1 - \alpha \cdot \alpha'}[M_{BA} + \alpha \cdot M_{AB}]$$

*JOURNAL, Amer. Concrete Inst., Nov. 1938; *Proceedings* Vol. 35, p. 93.
†Structural Engineer, Brooklyn, N. Y.

For the frequently occuring case of a uniformly distributed load on a member of constant I or any symmetrically haunched member:

$$REM_{AB} = -\frac{\alpha - \alpha \cdot \alpha'}{1 - \alpha \cdot \alpha'} \cdot \frac{WL}{4} \; ; \; REM_{BA} = \frac{\alpha' - \alpha \cdot \alpha'}{1 - \alpha \cdot \alpha'} \cdot \frac{WL}{4}$$

For a sidesway or other movements of supports as for example when support B is lowered as distance Δ:

$$\phi_{AB} = \phi_{BA} = \frac{\Delta}{L} \; ; \; M_{AB} = M_{BA} = -\frac{6KE}{C_1} \cdot \frac{\Delta}{L}, \text{ and}$$

$$REM_{AB} = -\frac{\alpha + \alpha \cdot \alpha'}{1 - \alpha \cdot \alpha'} \cdot \frac{6KE\Delta}{C_1 L} \; ; \; REM_{BA} = -\frac{\alpha' + \alpha \cdot \alpha'}{1 - \alpha \cdot \alpha'} \cdot \frac{6KE\Delta}{C_1 L}$$

Design tables of the type Table 1 may save labor even for such simple equations as (1) and (2). The value U added by this writer expresses the location of the zero point on the moment diagram Fig. 18, where same is loacted at a distance UL from the left support $\left(U = \frac{\alpha}{1 + \alpha}\right)$. A similar value, V, may be obtained for the right support. $\left(V = \frac{\alpha'}{1 + \alpha'}\right)$. These fixed points coefficients (U and V) afford a rapid graphical solution for the REM values as follows:

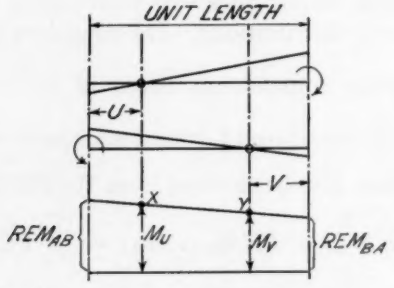

FIG. 23

Make $M_U = U \cdot M_{AB}$ and $M_V = V \cdot M_{BA}$. A straight line drawn through points x and y will give the required REM values as shown. This procedure is of particular value in all cases where the moment diagrams are being sketched anyway.

The authors' formulas (6) and (7) are not suitable for tabulation. This writer has overome this difficulty as follows:

TABLE 1

$\frac{K}{\Sigma\alpha}$	α	$\frac{\varkappa}{K}$	$U = \frac{\alpha}{1+\alpha}$
3.00	0.200	0.555	0.167
2.96	0.202	0.556	0.168
2.92	0.204	0.557	0.169

TABLE 2

J	UF or U	Q for F = 1	$\alpha = \frac{U}{1-U}$
1.49	0.167	1.11	0.200
1.48	0.168	1.11	0.202
1.46	0.169	1.11	0.204
0.80	0.217	1.16	0.277
0.39	0.265	1.22	0.361

Instead of C_1, C_2 and C'_2, tabulate (or chart) $S = \frac{3}{C_1 + C_2}$; $F = \frac{1}{S \cdot C_1}$

$S' = \frac{3}{C_1 + C_2'}$, and $F' = \frac{1}{S' \cdot C_1}$

Then: $UF = \frac{1}{3 + 2J}$, where $J = \frac{KS}{\Sigma R}$

$R = 2\varkappa = KS' \cdot Q'$, where $Q' = \frac{2F'}{3F' - \frac{1}{1-U}}$

(For constant I, $F = S = 1$)

This procedure would not have any particular merit if each item had to be solved. This is not the case however. Table 2, in conjunction with a chart for Q, F, U values gives immediate results as for example (Fig. 13): $r = 0.25$; $t_1 = t_o$; $\Sigma R_A = 2\Sigma\varkappa_A = 1.80$; $K = 1$

Then $KS = 1.45$, $F = 0.82$, $J = \frac{KS}{\Sigma R_A} = \frac{1.45}{1.80} = 0.805$

From table 2, $UF = 0.217$; then $U = 0.265$, $\frac{U}{1-U} = \alpha = 0.360$.

From Q,F,U diagram: $Q' = 1.49$; $R = KS'Q' = 1.45 \times 1.49 = 2.16$. Table 2 is of equal value to members of constant or variable moments of inertia.

The charts of the type Fig. 10 to 16 are often misused. They apply to rectangular sections only and not for T-sections.

The writer is looking forward to the day when one universal sign convention is adopted. He hopes that a moment that tends to sag a beam or cause tension in the bottom of the beam will be considered of positive sign and that the corresponding slopes of the elastic line at support will be considered of positive sign. This provides a perfect harmony between moment diagrams and calculations at any section of a member.

BY EDWARD L. AGRAMONTE*

Professor Wise presents in a condensed and clear manner a method of moment distribution with which, once having prepared a set of coefficients, final moments are obtained immediately for any loadings desired.

The advantage of writing down distribution and carry-over factors corresponding to a joint at the far end of each adjoining member, if any, is not explained. Convenience of having the carry-over factors well situated seems to be overbalanced by the disadvantage of having to search for the distribution factors far from the joint where they must be added.

For central spans of continuous prismatic beams with many equal spans, the precise factors become:

$$\alpha = 2 - \sqrt{3} = 0{,}26795. \qquad \varkappa = \frac{\sqrt{3}}{3} = 0{,}57735.$$

And for symmetrical loads:

$$REM = \frac{3\alpha}{1 + \alpha} FEM = 0{,}634\ FEM$$

These values are practically, for all but the last three spans at each end, exact.

There is plenty of literature, including useful diagrams, on haunched beams based on the ordinary stiffness and carry-over coefficients; their relation to the "precise" coefficients is hereafter explained:

Referring to Figs. 17 and 18 in the paper:

Let: I_o = Mom. of inertia of haunched beam at some convenient point.

$$K = \frac{I_o}{L} \qquad M_A = \alpha M_B \qquad M_B = \alpha' M_A$$

Eq. (9) extended to haunched beams reads:

$$M_B = \frac{6EK}{C_2 - C_1\alpha}\theta_B \quad \ldots\ldots (20)$$

and for the other end:

$$M_A = \frac{6EK}{C'_2 - C'_1\alpha'}\theta_A \quad \ldots\ldots (21)$$

The moment necessary to rotate B one unit ($\theta_B = 1$) could be termed "precise stiffness" at B and amounts to:

$$\sigma = \frac{6EK}{C_2 - C_1\alpha} \quad \ldots\ldots (22)$$

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Now let: $S = s \cdot EK$ = Moment necessary to rotate B one unit when A is fixed (ordinary stiffness at B).

s = Numerical stiffness depending on amount of haunching. (= 4 for prismatic beams).

r', r = ordinary carry-over factors from A to B and from B to A respectively, when far ends are fixed.

$C = \dfrac{1 - r'\alpha}{1 - r\,r'}$ = the Cross correction to take into account actual conditions at the far end.*

Then: $\sigma = \dfrac{S}{C} = \dfrac{s\,EK}{C} = \dfrac{EK}{\dfrac{C}{s}}$ which compared to Eq. (22)

implies that: $C_2 - C_1\alpha = \dfrac{6C}{s} = \dfrac{6}{(1 - r\,r')s} - \dfrac{6\,r'\alpha}{(1 - r\,r')s}$

Whence:

$$C_2 = \frac{6}{(1 - r\,r')s} \quad \ldots\ldots (23)$$

and $C_1 = r'\,C_2$ $\ldots\ldots (24)$

Calling s' the numerical stiffness at A, keeping section I_o as basis: it is known that $r\,S = r'\,S'$ which divided by EK: $r\,s = r'\,s'$ so that:

$$C'_2 = \frac{6}{(1 - r\,r')s'} = \frac{r'}{r}\,C_2 \quad \ldots\ldots (25)$$

and $C'_1 = r\,C'_2 = r'\,C_2 = C_1$ $\ldots\ldots (26)$

AUTHOR'S CLOSURE

Mr. Berg raised several questions. First, the use of Fixed End Moment as a starting point in the analysis is not essential to the method, since, as Mr. Berg point out, the angular changes at the ends of equivalent simple spans may be used. The amount of computation involved in these two procedures is practically the same, and therefore it would seem to be largely a matter of individual preference and not very important. Mr. Berg's solution and its relation to the use of Fixed End Moments is very interesting.

The tables to which Mr. Berg refers are contained in a book published by him.† This is a comprehensive handbook for the analysis of continuous structures and is a very fine treatise. It is so concise and its arrangement is such, that it would be necessary for an engineer to devote some time to becoming thoroughly familiar with it, in order to

*"Continuous Frames of Reinforced Concrete"—Cross-Morgan, p. 118.

†"Structural Bending Moments Simplified," by Ulrik T. Berg, published by Author, 1938.

be able to use it expeditiously. For one dealing with such problems constantly this would be well worth while.

The charts, Figs 10 to 16 of the original paper are for beams of rectangular cross-section, but have been used for cases where a tee section occurs near the center of span, because we have insufficient information concerning the behavior of such beams to enable us to determine their elastic properties more closely. In the absence of better information, the accepted practice, to use these charts is considered valid.

With reference to the sign convention, it is the Author's opinion that here again we have a question of personal preference. It is not particularly important and the results and equations in the original paper may be easily transformed so as to conform to the so-called "beam" convention of signs. There is no lack of harmony between moment diagrams and calculations in the convention of signs used in the original paper, if we remember that the "moments" referred to therein are couples external to the beam imagined as cut close to the supports. Then the convention of signs agrees with the one usually adopted in statics and enables us to apply the equations of statics without altering our signs. When we draw the moment diagram we need only interpret our moments in terms of their tendency to produce "sagging" or "hogging" moments. Perhaps it would be clearer if we designated moments in a beam as "*s*" (sagging) moments and "*h*" (hogging) moments, since the + and − signs usually used have that significance.

With reference to Mr. Agramonte's discussion, the distribution factors can be placed at the joint to which they refer if one prefers. It is only necessary to place the $\varkappa$-value at the far end of the beam from the α-value from which it is derived. The relationship between "precise" coefficients and those for fixed end conditions already published for beams of varying cross-section is very useful.

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN . . . . . NOVEMBER, 1938

Job Problems and Practice

Some will ask questions — Some will answer them — Some will do both

A. C. I. members are invited to use this new JPP department as an informal means toward mutual help. Some will ask questions; some will try to answer them; some will do both. When a problem holds possibilities of general interest the discussion will be briefed in these pages.

If you have a problem—present it; a question, ask it. If you know the answer to a question asked, or if you can contribute something which may help in the solution, or have reason to disagree with the answers or suggestions published, your contribution will be welcome.

If you know of an interesting problem whose solution has already smoothed the way of someone in the field, tell us about it—some other A. C. I. member may need just that information.

Why do test cylinders crumble? (No. 351)

Q—We are having trouble with test cylinders failing by crumbling at one end. This type of failure has only recently been experienced and is confined entirely to 7-day tests. Can you suggest the cause?

A—The fact that the trouble is confined to 7-day tests and has only recently appeared would suggest that it might be related to temperature for it is not unusual to experience during the early fall and winter difficulties with test cylinders made and cured in the field where special care is not exercised to see that these cylinders are maintained at standard curing temperatures. Crumbling such as your correspondent has experienced has been traced to leaving freshly made cylinders standing on a cold floor during the first two or three days after moulding. Another case was found to be due to cylinders only partially protected so that one end was maintained at a tempera-

ture four or five degrees different than the other. This sort of thing is likely to happen on jobs where of necessity the cylinders have to be stored during the first two or three days in field offices which, if heated at all in the early fall, will only be maintained at anything near the specified curing temperatures for a few hours a day.—R. B. YOUNG.

Why did the ceiling slabs pop? (No. 352)

On one of the government's housing projects some very serious popping and spalling of the concrete took place over a large area of concrete ceiling slabs. There were a number of casual suggestions as to the cause. One, a not uncommon explanation, was "frozen concrete." Another was that trucks had picked up mud on their wheels and that the mud had dropped off the wheels into the mixer skip. Government investigators went seriously at it and discovered that the popping was the result of small pieces of dead-burned dolomite which had found their way into the concrete by way of freight cars which had alternated the hauling of such material with the hauling of cement to the housing job. The cars not having been cleaned very well some of the dolomite was carried over with the cement.

Increasing block production without adding equipment? (No. 353)

Q—A manufacturer of standard concrete building units facing a need to increase the capacity of his plant, does not feel justified in making the expenditure for pallets and racks since the necessity for the increased production might be of short duration. He asks: "Has anyone experience or data to justify re-using pallets in less than a 24-hour period with ordinary curing methods, not high pressure steam?"

A—We have found it possible in making block, using a high early strength cement, during the early morning, to take the block off the pallets eight hours later if we put considerable heat into the curing room. This made it possible to re-use the pallets and racks the same afternoon and thereby gave us greater production.

In other words, an 8-hour cycle is sufficient if good curing is provided with a rich enough mixture of high early strength cement. The fact that the block were made in the latter part of August and the early part of September with good subsequent out-door curing conditions, naturally has a beneficial effect.

The yield obtained was twenty 12-in. block per 94 lb. of high early strength cement. Block made in the morning and taken off the pallets

in the afternoon were not actually tested for strength but block tested from similar batches which stayed in the curing room the full 24 hours, *indicated* strengths of 900 p.s.i. at the end of five days for the 8-hour cure.

It so happened that we were introducing dry heat into the kiln rather than moist heat. This may be one of the reasons for our ability to remove the block from the pallets the same day because we know that a wet block has less strength than a dry block. The block made in the morning and taken off the pallets in the afternoon couldn't be handled roughly but they were hard enough to be piled ten high without breaking corners. —BENJAMIN WILK.

The use of dry heat for speeding up production might be open to discussion—as not likely *always* to give satisfactory results. Who will say?—EDITOR

Cracks in bridge decks? (No. 354)

Q—A general checking of the surface of a concrete bridge deck in one of our northern states suggests that the fine cracks are due to the high shrinkage of the concrete in its plastic stage. These cracks are not prominent, except that after a rain, and when the surface is approaching its normally dry appearance, they are conspicuous. The question is, what effective treatment can be applied not only to minimize the unsightly effect from wetting, but to retard progressive deterioration of the surface?

A—(?)

Concrete tanks for formaldehyde solutions? (No. 355)

Q—We have noted that the Portland Cement Association, in its bulletins on "Effect of Various Substances on Concrete" states that aqueous solutions of formaldehyde (formalin) disintegrate concrete. The writer would appreciate information from any source on the chemical and physical effects that such solutions may be expected to have on untreated or unprotected concrete. Information is desired regarding any instances of storage of formaldehyde solutions in concrete tanks or vats, particularly with reference to the nature and effectiveness of protective coatings.

As to the immediate problem: Is concrete a suitable material for a large tank for storage of fish specimens preserved in a solution consisting of one part formalin and eight parts of water? The formalin is known as "trade formalin" and is a saturated solution of formaldehyde gas in water, which gives about 40 per cent strength. Ten per cent of ethyl alcohol may be added to this mixture. We want a tank 24 by 9 ft. and 3 ft. deep with two partitions.

A—It is the general impression among concrete engineers that formalin is detrimental to concrete, and to a limited extent this has been verified by our own tests on small containers. However, we have found that a number of commercial coatings are effective in protecting concrete against the 40 per cent solution, and it may be assumed that the very considerably diluted formalin will not be any more injurious than the full strength solution.

One of the materials which has given perfect protection against formalin in our tests is Bakelite Varnish XV-1657. The Bakelite Corp. advises us that this is the same as Tuf-On, No. 74, Clear, prepared by the Wipe-On Corp., which is made also in gray, black or aluminum. The so-called clear varnish is transparent but of a light tan color.

The concrete tank should have a smooth finish and be allowed to surface-dry thoroughly before the varnish is applied. Honeycomb or surface voids should be filled as soon as forms are removed. We should recommend that the varnish be applied in three coats 24 hours apart, the first thinned with about 25 per cent Xylol, or the special thinner recommended by the manufacturers. —J. C. P.

A concrete floor in a candy factory? (No. 356)

An old, badly worn floor in a candy factory was replaced with concrete. It was observed that this concrete was not hardening properly, and after waiting a few days it was decided that it was not going to harden. The floor was replaced, using different concrete materials on the assumption that they were the cause of failure. In this second attempt the concrete again failed to harden, which suggested the possibility that sugar contamination was responsible for this failure, since very small quantities of sugar prevent the setting of cement. In a very careful check of the concreting operations it was brought out that buckets belonging to the candy plant were used to measure the mixing water for the concrete. The contents of these buckets showed traces of sugar. Before placing the concrete for the third time, the base was very thoroughly scrubbed with an excess of water and every precaution taken to prevent the cement or sand being contaminated by sugar. The floor laid last hardened normally and is now giving good service. —H. F. GONNERMAN.

Non-durable chert? (No. 357)

An engineer on a government project writes: "In connection with the design of concrete mixtures for an important flood control structure the writer has had occasion to consider the suitability of local coarse aggregates containing a high percentage of non-durable chert.

In the general region of the work it has been observed that concrete structures in which such aggregates have been used have developed within a few years numerous surface spalls or 'pop-outs.' The subject appears to be an important one. Nevertheless a search of technical literature has revealed nothing pertinent except for general statements that such material should be used with caution. What have you to suggest?" So far the Editor has not found anyone willing to go on record in an answer to this inquiry, but three references follow:

The Stability of Chert, by H. F. Kriege
Rock Products, April 27, 1929.
Chert Unfit for Coarse Aggregate in Concrete, by F. V. Reagel
Engineering News Record, Aug. 1924
Deleterious Substances in Concrete Aggregates, by F. C. Lang
Nat. Sand & Gravel Bulletin, April, 1931.

Further discussion of this subject is invited from those who seem to have found the answer.

How to place concrete for exposed surfaces? (No. 358)

Q—What is the best method of placing concrete in the walls of buildings where it is to be left exposed—that is, where an architecturally acceptable surface is to be obtained.

A—Special care must be exercised to prevent segregation of the concrete when it is placed in deep wall forms, otherwise unsightly stone pockets and rough arrises and bleeding may result.

It is also important to prevent splashing of the forms and reinforcement above the general level of the concrete being placed. Partially hardened concrete on the forms will result in blemishes in the finished surface.

It is relatively simple to prevent segregation and splashing of the forms and reinforcement. This is best accomplished by requiring that all concrete be placed through an elephant trunk or spout which extends down into the form so that there will not be a free fall of more than 3 ft. from the end of the spout to the level of the concrete already in place. Elephant trunks are made of about 20-gauge galvanized iron and the spout is rectangular in cross section. The dimensions of the section will depend upon the thickness of wall and the location of the reinforcement. In general, however, a spout about 4 by 12 in. is most satisfactory. Round metal and canvas spouts have been used but they are not so satisfactory because it is difficult to get the concrete through them in sufficient quantity and the canvas spouts catch and hang on the reinforcing. It is customary to make the spouts in sections about 2 ft. 6 in. long so the length can be adjusted as the concrete

rises in the forms. A typical detail for hopper and trunk for placing concrete in forms is shown (Fig. 1).

At all times during the placing of concrete there should be sufficient illumination inside the form so that the place of deposit is clearly visible from the deck above.

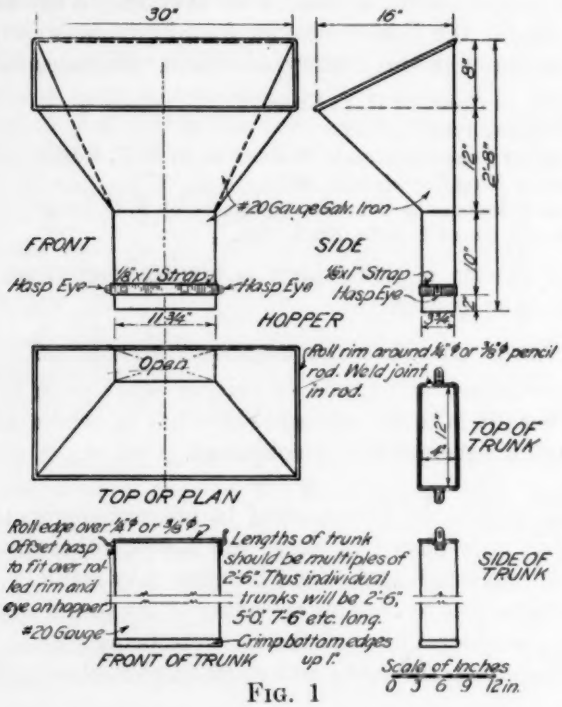

FIG. 1

Thorough spading of the concrete is of course essential and it is advisable to augment hand spading by rapping the forms with rubber hammers or wooden mallets or by light pneumatic or electric hammers (chipping guns). The practice is to apply the pneumatic or electric hammers to the studs and wales on the outside as nearly as possible at the level of the concrete being placed. This procedure will aid greatly in preventing stone pockets and will tend to work air out of the surface, thereby minimizing the number of air bubbles.—L. C.

Timing the Mix? (No. 359)

Q—Do you know of a simple method of timing a concrete mix?

A—An effective timer can be on the principle of an hour-glass made by using a water gauge glass and placing a small plug in the centre

with an orifice to permit a fine uniform grading of dry sand to pass through it in the time specified for the mix. A cork placed at each end retains the sand. This tube may be attached to a small board that would rotate around a bolt which is attached to another board; or a small hook could be taped to each end of the glass tube.

Probably a more satisfactory method of timing the mixer is to use a timing clock of the type used by photographers in making prints. These make one revolution in a minute and can be read at considerable distance, are quite rugged, and cost only a few dollars.—WILFRID SCHNARR

It should be emphasized that the man who faces a problem often can't wait for the ultimate solution. He can't go off the job until somebody cracks the last cylinder in a 5-year research program and meticulously analyses the data. His problem is current—today. He wants the best answer he can get ***now***. So, this JPP department is not for treatises. It will provide many fragments of which next year's treatises will take note. It will provide a place where ***A***, who is in a jam, meets ***B***, who was once in a similar jam and got out of it, or who knows somebody who was and did—and how.

The "answers" are the answers of individuals to whom the questions are referred and not of the A. C. I. as an organization.—EDITOR

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN NOVEMBER 1938

Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Structural materials and methods through the ages

RICHARD GRUN. *Tonindustrie Zeitung*, V. 62, No. 35, 37, 38, p. 393-5, 414-7 428-30, May 2, 9, 12, 1938.
Reviewed by A. E. BEITLICH

An illustrated historical review.

Burning of cement slurry on a grate

ERICH SCHIRM. *Tonindustrie Zeitung*, V. 62, No. 33, p. 367-72, April 25, 1938.
Reviewed by A. E. BEITLICH

A description of various types of installations with a discussion of their mechanical features, their particular merits and efficiencies.

The efficiency of air separation—2

JAKOB HEYD. *Tonindustrie Zeitung*, V. 62, No. 55, 56, 59, p. 593-5, 604-7 640-1, July 11, 14, 25, 1938.
Reviewed by A. E. BEITLICH

This is the second and final section of a comprehensive treatise on air separation. See the review on page 709 of the May-June 1938 issue of this journal.

Bending trussed bars

WILLIAM SELL, *Engineering News-Record*, Vol. 121, No. 11, Sept. 29, 1938, p. 414.
Reviewed by ROBT. W. BEAL

A nomograph is given which will be found useful by reinforced concrete detailers when laying out bent bars for slabs and beams.

Reinforced concrete chimneys

C. P. TAYLOR and LESLIE TURNER, *Concrete and Constructional Engineering*, V. 33, No. 9, Sept. 1938, p. 449-453.
Reviewed by GLENN MURPHY

This installment of the series contains the calculations involved in the design of a section of a 275 ft. chimney. The effect of wind loading and temperature is included.

The development of foundation stabilization through chemical means

Beton und Eisen, Vol. 37, No. 14, p. 225, July 20, 1938 Reviewed by A. U. TREUER

An editorial review of a lecture by Adolph Mast describing the application and field practice of foundation stabilization by chemical means as developed by Doctor Joostens. Many illustrations show the method as applied in the field.

Two camera hints on bridge paving

Engineering News-Record, Vol. 121, No. 9, Sept. 1, 1938, p. 279 Reviewed by S. J. CHAMBERLIN

Metal lath instead of the usual tight wood panel was used for the bottom form of the concrete sidewalk slab. A turn table set on a steel crossbeam of the bridge floor system was used to turn the concrete trucks for rear dumping.

Bending knee reinforcing

W. C. GOODWIN, *Engineering News-Record*, Vol. 121, No. 8, Aug. 25, 1938, p. 245.
Reviewed by S. J. CHAMBERLIN

A method is given for determining the radius of bend and the points of curvature of the reinforcing bar around the knee of a rigid frame, and for having the bend join the two tangents within a specified clearance.

The determination of the fineness of cement with the turbidimeter

H. ELSNER VON GRONOW. *Tonindustrie Zeitung*, V. 62, No. 33, p. 361-5, April 25, 1938.
Reviewed by A. E. BEITLICH

A turbidimeter similar to that developed by Klein for the determination of the specific surface of portland cement and raw mix by means of photoelectric measurements is described in detail.

The hingeless parabolic arch

R. J. CORNISH. *The Structural Engineer*, July 1938, p. 224. Reviewed by N. H. ROY

This paper deals with the effect of settlement of the abutments of an arch. Bending moments, horizontal thrust and shear for a hingeless arch of any shape are obtained from strain energy considerations. This paper is a continuation of the excellent first paper by the same author, reviewed in Mar-Apr. JOURNAL, p. 529.

Accelerated test for the soundness of hydraulic lime

EBERHARD SPOHN. *Tonindustrie Zeitung*, V. 62, No. 45/46, p. 501, June 9, 1938.
Reviewed by A. E. BEITLICH

One hundred grams hydraulic lime are mixed with 12 cc. of water and pressed in a mold of 80 mm. diameter for 1 to 2 minutes with 20 ton pressure. The specimen is then placed with free access of air in a dryer at 80°C. After 2 hours the specimen should be free from cracks and not warped.

Phase composition of cement clinker—part 1

L. A. DAHL, *Rock Products*, Sept. 1938, Vol. 41, No. 9, pp. 48-50 Reviewed by ROY N. YOUNG

This is the first part of a treatise to be published through several issues of *Rock Products* on phase compositions in the system $3CaO.SiO_2-2CaO.SiO_2-3CaO.Al_2O_3-4CaO.Al_2O_3.Fe_2O_3$ at clinkering temperatures. Principles in the calculation of phase composition are discussed and examples are given.

German specifications DIN E 1165 describing the apparatus for making and testing 4 by 4 by 16 cm. prisms of plastic mortar

Tonindustrie Zeitung V. 62, No. 76, p. 850-2, Sept. 22, 1938. Reviewed by A. E. BEITLICH

Proposed specifications giving detailed descriptions and drawings of the molds, tampers, measuring plugs, flow table, curing chamber, bending test apparatus, compressive strength test and shrinkage test apparatus.

Charts for rapid design of slabs

L. W. ASHURST, *Concrete and Constructional Engineering*, V. 33, No. 8, Aug. 1938, p. 410-414. Reviewed by GLENN MURPHY

Six charts are given for the design of floor slabs based on a uniform load of 62, 74, 92, 104, 112, and 124 lb. per sq. ft. plus the weight of the slab, and two charts for roof slabs carrying uniform loads of 92 and 104 lb. per sq. ft. Each chart gives the required slab thickness, and the size and spacing of reinforcement for moment coefficients of $\frac{1}{12}$, $\frac{1}{10}$, and $\frac{1}{8}$ at each end. The charts are based on S_c = 750 p.s.i., f = 18,000 p.s.i. and n = 15.

Reinforced concrete station roofs

Concrete and Constructional Engineering, V. 33, No. 8, Aug. 1938, p. 405-409. Reviewed by GLENN MURPHY

Description of two 200 ft. station roofs on a new electrified branch of the Southern Railway. The 3-in. curved roof slab is supported by a series of cantilevered ribs with an overhang of 14 ft. The design of the roof slab is unusual in that one layer of reinforcement follows the direction of the lines of principal tensile stresses. The other layers are placed in the conventional manner.

Viaduct built from barges

Engineering News-Record, Vol. 121, No. 11, Sept. 15, 1938. pp. 329-331. Reviewed by ROBT. W. BEAL

Details of construction methods are given in connection with building the new Galveston, Texas, causeway with equipment to perform many repetitive operations over the two mile distance. The general plan involved dredging a parallel canal for a light, fast-moving barge plant which served as the base of operation. The light equipment was chosen because of the many small pours, and the many moves required to place them.

Three-story reservoir of novel construction

Engineering News-Record, Vol. 121, No. 8, Aug. 25, 1938. p. 230 Reviewed by S. J. CHAMBERLIN

A reinforced concrete reservoir, consisting of thin arch sections supported by beams and buttresses, was recently completed at Nantes, France. The lowest basin (243 ft. in diameter) consists of 50 thin arches inclined at an angle of 45 degrees. The sides of the middle basin are thin vertical arches, and the bottom is formed by segmental arches which extend to form the roof of the lower basin. Similar construction was used for the top basin.

Control silica dust and sand gradation problems—producing aggregates for hiwassee dam

STAFF EDITOR, *Rock Products*, Sept. 1938, Vol. 41, No. 9, pp. 28-34 Reviewed by ROY N. YOUNG

The equipment and procedures for producing stone aggregate and stone sand for concrete in the erection of the Hiwassee Dam are described in considerable detail. One of the features of this plant as compared with others of the T. V. A. is the provisions for dust control. Concrete will be placed at an average rate requiring 4000 tons of aggregate per day with a maximum of 5000 tons. The plant was designed by the U. S. Engineers to produce 325 tons hourly with a maximum of 400 tons.

Progress of cement investigation in 1937

C. R. PLATZMANN, *Zement*, Vol. 27, No. 32, 33, 34, Aug. 11, 18, 25, 1938, pp. 483-488, 499-504, 512-517. Reviewed by L. T. BROWNMILLER

A comprehensive review of the literature published during 1937, covering all phases of cement and clinker investigations, manufacture, burning, hydration, durability, etc. Each paper reviewed is condensed into a few sentences so that the value of the report is in its references, particularly the Russian.

The macrostructure of portland cement clinker

B. TAVASCI, *Tonindustrie Zeitung*, V. 62, No. 72/73, p. 794-5, Sept. 12, 1938. Reviewed by A. E. BEITLICH

A method is described in which polished clinker sections are etched and observed with relatively low-power magnification. The different structures of the clinker can be studied, especially how the structure is affected by various methods of manufacture. It is pointed out that this method facilitates an estimation of the different clinker constituents. The etching media for the differentiation of alit and belit were 1 per cent solution of borax and a 1 per cent solution of nitric acid in isoamyl alcohol.

The cooling system adopted in the construction of the Saale-Valley Dam

DR. ING. RAU, *Beton und Eisen*, Vol. 37, No. 16, p. 257 Aug. 20, 1938. Reviewed by A. U. THEUER

This is a brief description of a cooling system used in the construction of the Saale Valley Dam, a curved gravity structure 246 feet high and containing 610,000 cu. yds. of concrete. The cooling plant, an ammonia system, is stated to have a capacity of 1.2 million kilogram-calories per hour, or a capacity for cooling 350 cu. meters of Saale river water 4°C each hour. This is stated to be the first German application of a cooling system in dam construction.

Colored cements and their application

Tonindustrie Zeitung, V. 62, No. 33, p. 373-4, April 25, 1938. Reviewed by A. E. BEITLICH

White cement is used as basis for most colored cements to obtain pure and clear colors. In many cases the clinker is mixed with the color before being ground to insure even distribution. Commercially made are pink, terracotta, yellow, brown, violet, black, orange, blue, green, and cream colored cements. Red-brown cements can be made with a gray cement. There is very little danger that the color affects the physical properties of the cement. The various fields for the use of colored cements are described.

The Rudersdorf accelerated soundness test for hydraulic slaked lime

G. FRENKEL. *Tonindustrie Zeitung*, V. 62, No. 34, p. 381-3, April 28, 1938 Reviewed by A. E. BEITLICH

A paste which is prepared from 5 cc. of water and 35 g. of powder is compressed in a mold of 5 cm. diameter for one half minute at 1000 kg. pressure. The specimen is then exposed to water vapor of 65°C., and the expansion is measured with the Le Chatelier apparatus. The sensitivity of the method is 1 to 2 per cent unslaked lime. A relation has been observed between the disintegration in long-time tests and the degree of expansion found in the accelerated test.

New hurricane bridge design

Engineering News-Record, Vol. 121, No. 11, Sept. 15, 1938, pp.. 327-328 Reviewed by ROBT. W. BEAL

Galveston, Texas, has experienced difficulty in maintaining highway connections between its island location and the mainland. Previous structures have been constructed with the idea that strength of mass was the best insurance against damage from hurricanes and tidal waves. A replacement program now in progress is employing an open type design to reduce lateral impact of storm waves. However, there is provided sound structural resistance, with the complex forces of cyclonic storm wave action being taken into consideration.

Raw material emphasized in reclamation bureau—concrete control

JACK HAVARD, *Rock Products*, Sept. 1938, Vol. 41, No. 9, pp. 35-38 Reviewed by ROY N. YOUNG

This article tells a story of the workings of the U. S. Bureau of Reclamation that are focused in the concrete and cement laboratories at Denver. One outstanding aim is excellent concrete. This, for any one project, calls for the solution of numerous problems from the prospecting for suitable concrete aggregate to the never ending observations on the finished structure. Some of these problems are briefly discussed and some of the achievements of research are mentioned. The more unique or important pieces of equipment are described. Much information is crowded into this short article, yet it is highly readable.

Analytical investigation of hardened concrete

A. STEOPOE. *Tonindustrie Zeitung*, V. 62, No. 33, p. 365-7, April 25, 1938.
Reviewed by A. E. BEITLICH

From a series of studies of hardened concrete the following conclusions were drawn. In the case of a concrete containing practically insoluble aggregates, the proportion of the mix can be estimated by means of three methods, by determining the residue insoluble in hydrochloric acid, the soluble amount of silica, and the soluble amount of lime. The fine fractions of the aggregates affect the properties of the cement not only because of their fineness, but also on account of their chemical nature which needs to be taken into consideration.

Chemical factory at Welwyn Garden City

R. A. MANTHEI, *Concrete and Constructional Engineering*, V. 33, No. 9, Sept. 1938, p. 457-467.
Reviewed by GLENN MURPHY

Design features of the recently-completed factory of Roche Products, Ltd. The factory includes an office block, a laboratory block, a warehouse, factory, and sheds. All inside pipes in the treating system are cased in the floors, walls or columns. To provide sound insulation between floors, the concrete floors were covered with a 1½ in. layer of compressed cork and a layer of cardboard. A 2-in. "floating" slab of reinforced pumice concrete was then poured on the cardboard to form the floor. The floating slabs are also separated from the walls by a layer of compressed cork.

Reinforced concrete chimneys

C. PERCY TAYLOR and LESLIE TURNER, *Concrete and Constructional Engineering*, V. 33, No. 7, July 1938, p. 351-370 Reviewed by GLENN MURPHY

This article, the fourth in a series, deals with the calculation for temperature stresses in reinforced concrete chimneys. Equations, based on the assumption that the temperature gradient is a straight line through the concrete shell, are developed for the following cases: temperature stresses alone, temperature stresses combined with vertical compression with steel in tension or compression, temperature stresses combined with vertical tension, and horizontal stresses. Both the conditions of one ring of vertical steel (near the outside of the shell) and two rings of vertical steel (one near each face) are considered.

Subzero concrete pumping

F. A. DALE, *Engineering News-Record*, Vol. 121, No. 13, Sept. 29, 1938, pp. 407-409
Reviewed by ROBT. W. BEAL

At the Solon dam in northern Maine, concrete was pumped through 280 ft. of non-insulated pipe, with air temperatures as low as -20°F. The pipeline was prewarmed by steam before pumping was started. The delivery of concrete to the form

by pipe facilitated the overhead covering of the forms so that the entire form compartment could be heated. It was found that pumping could be stopped for 20 minutes without plugging, and that the general behavior during transportation was dependent on the shape and grading of aggregates as well as on mix proportions. Slumps as low as 1½ in. were pumped without difficulty. The cost of heating aggregates and concrete, before and after placing, amounted to $0.39 per cu. yd.

Measuring concrete vibrations

Engineering News-Record, Vol. 121, No. 5, p. 141, Aug. 4, 1938. Reviewed by S. J. CHAMBERLIN

A short description of the apparatus developed by the National Bureau of Standards to measure the particle displacements, velocities and accelerations set up by a vibrator inserted into a concrete mass. The apparatus is said to be useful in determining the efficiency of a vibrator, and the effect of the different variables in vibrated concrete. A small electromagnetic pick up unit immersed in the concrete produces an alternating voltage which is proportional to the velocity of the concrete motion in a given direction. The apparatus is described in detail by George L. Pigman, F. B. Hornibrook, and J. S. Rogers in the May 1938 *Journal of Research* of the Bureau.

Grading concrete aggregates

H. N. WALSH, *Concrete and Constructional Engineering*, V. 33, No. 8, Aug. 1938, p. 421-432 and No. 9 Sept. 1938 p. 471-480 Reviewed by GLENN MURPHY

In the first installment of an article on the grading of the aggregate for concrete mixtures, sieve analysis curves are given for the gradings to produce the concrete of maximum density from gravel and crushed stone aggregate of ¾ in. maximum size. Separate curves are given for each of the aggregates in the following proportions: 1:2½:3, 1:2:4, 1:2½:5, and 1:3:6. Later articles will deal with aggregates of different maximum size.

The second installment presents the grading curves for stone and gravel aggregates to produce a concrete of maximum density where the aggregates have maximum sizes of 1½ in., 2 in., and 3 in.

Dam tested by artificial earthquakes

Engineering News-Record, Vol. 121, No. 6, Aug. 11, 1938, pp. 184-185. Reviewed by S. J. CHAMBERLIN

An oscillator was used on the Morris Dam to check on its computed period of vibration. The concrete dam of gravity section has a vertical joint containing several offsets to give sliding contact faces parallel to the direction of movement believed to have occurred in an old fault. The computed period of vibration was about 0.14 sec., while the machine produced two resonance peaks, one at 0.17 and the other at 0.20 sec. The tests showed that the two parts of the dam separated by the joint act together in resisting vibrational forces. In many cases the amplitudes of vibration were larger on the side of the joint away from the oscillator. For this structure the amplitude of vibration was proportional to the vibrating force.

The hardening problem of hydraulic cements

V. RODT. *Tonindustrie Zeitung*, V. 62, No. 17, p. 188-90, Feb. 28, 1938.
Reviewed by A. E. BEITLICH

The products formed in the reaction between cements and water consist of (1) distinct microscopically visible crystals as in the case of hardened gypsum, (2) extremely fine particles which appear amorphous but which exhibit definite x-ray patterns as in the case of hardened lime or Sorel cement, or (3) colloidal particles or crystals formed from originally colloidal matter. In the hydraulic hardening process

it is of little importance whether colloidal or crystalline products are formed. The most essential requirement is that the product which is formed in the mortar, takes up so much water that only little of the mixing water remains free. It is also important that the hardened product does not take up additional appreciable amounts of water.

Sodium sulphate-freezing and thawing tests reveal effect of deleterious materials in concrete

R. R. Litehiser, *Rock Products*, Sept. 1938, Vol. 41, No. 9, pp. 39-40 Reviewed by Roy N. Young

The following substances are considered deleterious to concrete when contained in Ohio gravel aggregate: shale; limonite, or yellow oxide of iron (a common ingredient in so-called chocolate bars or nigger toes); hematite; pyrites; and one kind of chert which is white or gray colored, non-waxy and absorbent. The effects of such substances are illustrated by photographs of both laboratory specimens and concrete structures.

Freezing and thawing tests carried on over a period of two and one-half years show that present accepted specification limits covering deleterious materials cannot be made less exacting.

Relation of compositions and heats of solution of portland cement clinker

Herbert Insley, Einar P. Flint, Edwin S. Newman and J. Arthur Swenson, *Research Paper RP1135* National Bureau of Standards, Vol. 21, No. 3, Sept. 1938 Author's Abstract

The results of chemical and microscopic analyses and heat-of-solution measurements on samples of commercial portland cement clinker are reported. Differences between quantitative microscopic analysis and compound composition calculated from chemical analysis have systematic relationships with chemical composition of the clinker. The heats of solution calculated from compound composition by microscopic and by chemical analysis show fair correspondence with the observed heats of solution. Assumptions on which both methods of calculation are based are shown to be somewhat faulty.

Handling corners in rigid frames

B. J. Lambert and C. J. Posey, *Engineering News-Record*, Vol. 121, No. 5, Aug. 4, 1938, pp. 147-149. Reviewed by S. J. Chamberlin

Design suggestions are given on a troublesome but vitally important element of the popular rigid frame. The rounded inside and outside corner of a reinforced concrete frame will have either tension or compression on the inside depending on the loading. Tension at the inside will require stirrups to prevent the main reinforcing bars from pulling away on the inside and pushing away on the outside. The stirrups must be securely attached to the main bars. Stirrups at the outside of the bar are also necessary to keep the main bars from buckling. Frames with sharp corners having tension on the inside of the bend should have the inside bars extending through to the far face, and have additional diagonal bars. The outside compressive steel should be bent to a radius and be provided with stirrups.

Phase equilibria studies on mixtures of the compounds $4CaO.Al_2O_3.Fe_2O_3$-$2CaO.Fe_2O_3$-$K_2O.Al_2O_3$

William C. Taylor, *Research Paper RP1131*, National Bureau of Standards, Vol. 21, No. 3, Sept. 1938 Author's Abstract

A portion of the system $CaO-Al_2O_3-Fe_2O_3-K_2O$ has been investigated. The quenching method of study of phase equilibria was employed. No ternary compounds containing K_2O were observed. The compounds $K_2O.Al_2O_3$ and $4CaO.Al_2O_3.Fe_2O_3$ were found to form a binary system with one eutectic. Since it was observed that,

for a given ratio of $4CaO.Al_2O_3.Fe_2O_3$ to $2CaO.Fe_2O_3$, the solid solution of these two compounds and $K_2O.Al_2O_3$ resembled a binary system with a single eutectic, these compounds are considered as forming a pseudobinary system. Pertinent optical and thermal data have been presented. Temperature-concentration diagrams for sections of the quaternary system are constructed.

Lateral earth and concrete pressures

LAZARUS WHITE and GEORGE PARASWELL, *Proc. Am. Soc. C. E.*, Vol. 64, No. 7. (Sept. 1938) p. 1319-1334 Reviewed by H. J. GILKEY

Recognizing the unsatisfactory results procurable by attempting to build analyses upon or about the classical work of Coulomb and Rankine, the authors have instead developed their analysis from the fundamental laws of the theory of elasticity which does result in a somewhat involved mathematical treatment. Theoretical results were compared with observed form pressures on the 20 ft. high concrete piers of the Alton (Illinois) Dam on the Mississippi River. The observed maximum form pressures were between 700 and 800 lb. per sq. ft. for heights of concrete between 10 and 20 ft. These decreased to about 100 lb. per sq. ft. at a height of 2 ft. The calculated maximum pressure was slightly in excess of the maximum observed at the 8 ft. height but this decreased each way from the maximum.

Method for the computation of slabs in which the moments of inertia are unequal

V. A. NICOLSKY, *LeGenie Civil*, Vol. CXII, No. 22, May 28, 1938, 456-457 Reviewed by R. L. BERTIN

The classic treatise of thin plates are based on the plates being uniform in thickness and homogeneous, from which follows that the Moments of Inertia in any direction are equal.

Rectangular reinforced concrete slabs, because of unequal reinforcements, do not comply with this premise.

The author derives a method of reconciling the use of design methods based on the elastic theory for rectangular reinforced concrete slabs through the use of a substitute slab in which the span along the "x" axis is modified by the factor $\frac{\sqrt[4]{I_y}}{I_x}$

Small shields drive tunnels for Staten Island sewers

Engineering News-Record, Vol. 121, No. 5, Aug. 4, 1938, pp. 149-150 Reviewed by S. J. CHAMBERLIN

The shield driven tunnels are lined with tight-joint, precast, trapezoidal blocks erected during driving operations. The tongue-and-groove joints are sufficiently watertight for drainage purposes without secondary lining. Twenty-four blocks make up a ring for the 6-ft. tunnel, and 18 for the 4-ft. one. The blocks, with inner and outer faces curved, are divided into two types, pushers and fillers, which occupy alternate positions in a complete ring of lining. The pushers bear the brunt of the shield shoving. A complete ring is fully interlocked in itself as well as locked into the previously set ring by tongues and grooves. Contact faces, and the locking tongues and grooves are dipped in hot asphalt before setting. The blocks, made in a hydraulic press, are of a 1:1.8:2.2 mix. Required strengths of 2500 p.s.i. at 7 days, and 3500 p.s.i. at 28 days were surpassed.

The 41st General Meeting of the German Concrete Society in Berlin from March 8 to 10, 1938

MAUNE, *Tonindustrie Zeitung*, V. 62, No. 22, p. 249-51, March 17, 1938. Reviewed by A. E. BEITLICH

A review of papers presented at the meeting including: Grün: "Structural materials through the ages," Wegenast: "The bridge of the German automobile

roads over the Danube River near Leipheim," Schleussner: "The reinforced concrete structures of the central airport Berlin-Tempelhof," Kischlat: "The construction site for the Congress Hall in Nuremberg," Rau: "The construction site for the Saale dam Hohenwartha," Gehler: "Hypotheses and fundamental considerations for the shrinkage and creep of concrete." Finsterwalder: "Reinforced concrete beams," Ernst: "The surface treatment of concrete and reinforced concrete structures," Dörr: "Modern reinforced concrete silo structures," Spaeth: "Concrete for fortifications and the effect of projectiles."

Centering hung from pier umbrellas

A. Burton Cohen, *Engineering News-Record*, Vol. 121, No. 5, Aug. 4, 1938, pp. 145-146.
Reviewed by S. J. Chamberlin

Construction of three concrete barrel arches of a bridge at Binghamton, N. Y., was aided by hanging the full-span steel arch centering on steel rods passing through short cantilever sections of the arches built as a continuation of the piers. Four hanger rods, at either end of the four trusses of each arch center, passed through 4-in. pipe sleeves in the pier umbrella to a bolted bearing on the extrados. The vertical load applied in cantilever, balanced the overturning moment of the horizontal thrust of the centering, when there was loading on one side of the pier only. When there was a completed arch on one side it was allowed to cure at least two weeks before the adjoining centering could be removed. A strain gage was used to establish a uniform initial stress in the hanger rods. After pouring, variations in load were found by the gage and corrections made by tightening the low-stressed rods.

Condensation of single-layered road concrete by vibration

F. Kaufman, *Die Betonstrasse*, V. 13, No. 8, pp. 159-162, Aug. 1938. Reviewed by F. L. Ehasz

A brief description of the "Vibromax AT 300" surface vibrator is given, as well as results of tests on a series of eight concrete slabs of 8-in. (20 cm.) and 12-in. (30 cm.) thickness. Operation of the machine has been found to be fairly expedient as a result of the inherent horizontal centrifugal force, acting in the direction of travel and controlled by the raising or lowering of its handle. For the 8-in. thickness a vibration period of 8.4 sec./sq. ft. (90 sec./m²) for single-layered slabs yielded a good dense product for both gravel and crushed-stone concrete. It is pointed out further that the strengths of the upper and lower surfaces of the 12-in. slabs were practically the same. A period of 12.6 sec./sq. ft. (135 sec./m²) in the case of 12-in. thickness is recommended for use at the construction site. The question of economy is briefly mentioned.

Air-raid shelters

Annonymous, *Concrete Building and Concrete Products*, Vol. 13, No. 9, Sept. 1938, p. 186.
Reviewed by J. C. Pearson

This brief article describes two types of air-raid shelters marketed by a progressive English manufacturer of spun concrete pipes. The larger of these, accommodating 50 persons, is made of ten pipe sections, each 90 in. in diameter and 5 ft. long, laid horizontally below ground level. Access is through smaller vertical pipes at the ends, fitted with ladders and gas tight covers. They may be equipped with gasketed doors, air-lock chambers, and toilets. A level floor about 18 in. wide is provided along the bottom and wooden seats are arranged along each side. The smaller shelter, accommodating 8 persons, is made of three 2 ft. sections of 72 in. pipe erected vertically on a concrete slab laid in the bottom of a pit, and roofed with a slab of concrete. Access is through a 27 in. diameter pipe, fitted with step irons and gas tight cover. The shelters are covered with earth to any depth desired. No reference is made to ventilation.

Modern building protection

C. R. PLATZMANN. *Tonindustrie Zeitung*, V. 62, No. 19, p. 214-6, March 7, 1938.
Reviewed by A. E. BEITLICH

Protective agents are divided into integral admixtures and protective coatings. Powders of the first group should be mixed with the dry cement. No integral admixture should lower the strength of the cement more than 10-15 per cent. Directions for the proper use of such admixtures are given. Of great effectiveness are bituminous coatings which have been thoroughly studied in recent years. They are applied on dry surfaces or in the form of emulsions on wet surfaces. Fluosilicate coatings provide protection due to a chemical reaction with the cement, the presence of lime being necessary. They are not immune against the attack of acids but give excellent protection against mineral oils and very weak organic acids. The best means for protecting concrete surfaces against chemical or mechanical attack is to make a dense concrete surface or to use hard durable aggregates. The final section of the paper deals with materials for the insulation against heat, cold and sound.

Light-weight concrete

ALFRED HUMMEL. *Tonindustrie Zeitung*, V. 62, No. 3, p. 31-3, Jan. 10, 1938.
Reviewed by A. E. BEITLICH

The article gives several excerpts from the chapter on light-weight concrete from the second edition of a recently published book "The concrete A B C." The various types of light-weight concrete are classified into (1) porous materials containing light or fibrous aggregates, (2) concrete made of coarse porous aggregate embedded in a relatively dense mortar, (3) concrete made of coarse dense aggregates cemented together at their points of contact, and (4) concrete as under (3) made however with porous aggregates. The proper selection of the type depends on the available aggregate and the desired concrete. A table is given showing the physical properties of various natural and artificial aggregates for light-weight concrete. The means for controlling the structure of the concrete are discussed. Special consideration must be given to the water-cement ratio since loss of water into the pores of the aggregate may affect the ratio considerably.

Comparative bending and compressive strength measurements on beams, prisms, and cubes of mortar and concrete

H. PASSOW, *Tonindustrie Zeitung*, V. 62, No. 72/73, p. 790-4, Sept. 12, 1938
Reviewed by A. E. BEITLICH

The bending and compressive strengths of the broken ends of prisms, used in bending strength tests, are affected by the size of the specimens. The bending strength data obtained with the standard prisms cannot be applied directly to concrete. Every cement reacts different. The same is true for the compressive strengths of the broken ends. It is concluded that the proposed 4 by 4 by 16 cm. prism is unsuitable for the determination of the bending strength of a cement as well as for the determination of the compressive strength by means of the prism ends. These tests do not portray the behavior of the cement in concrete. The test results obtained were less satisfactory than those reported by Gehler and Frank in Feb. 1938. The author recommends additional investigations, especially with specimens of larger dimensions (10 by 10 by 50 cm. concrete), approaching more closely the conditions in the field.

Crack control in concrete walls

A. M. YOUNG, *Engineering News-Record*, Vol. 121, No. 6, Aug. 11, 1938, pp. 178-179.
Reviewed by S. J. CHAMBERLIN

The reinforced concrete walls of a school building were built with dummy joints to prevent random and irregular cracks. The 9-in. walls are reinforced on both

faces, the outer reinforcement being 2 in. from the face of the wall. The dummy joints were placed in vertical planes from below grade to the top of the parapet. The joints are ¼ in. wide, 1½ in. deep on the outer face, and 1 in. deep on the inner face. Every alternate horizontal bar (⅜ in. on 8-in. centers) passing through the joint was cut. Before the color coat was applied to the exterior the grooves were filled with a mastic. After a year of service no cracks are visible on the exterior walls. Cracks do extend at the joint from front to back. Similar joints used in the interior corridor walls were centered over the door openings. In some cases cracking has occurred at the corners of the door opening in spite of the joints. The article prompts an editorial in the same issue entitled "Why Monolithic?"

Waterproofing agents for mortar and the limits of their effectiveness

C. R. Platzmann, *Tonindustrie Zeitung*, V. 62, No. 72/73, p. 802-3, Sept. 12, 1938.
Reviewed by A. E. Beitlich

The author defends the value of waterproofing agents and their place in the construction field against some attacks in the literature which are not based on properly conducted investigations. For a beneficial effect on any such agent it is necessary that a certain degree of density exist in the mortar or concrete and that there are enough chemically active cement compounds available. Trass acts as a pore filler and reacts with the lime liberated from the cement. Most waterproofing agents belong to the group of salts of fatty acids. They are either soluble in which case they form insoluble compounds with the hydrated cement, or they are insoluble and then act chiefly as filler. Lately a new type which has been used with success is a finely divided iron powder. Mixed with the cement paste, the iron rusts rapidly with a corresponding increase of about 2.2 times its original volume, thereby increasing the impermeability of the mortar.

Low cost water storage

E. A. Davis, *Engineering News-Record*, Vol. 121, No. 10, Sept. 8, 1938, p. 301-302
Reviewed by S. J. Chamberlin

The thin walls and dome roof of a circular reservoir were built up of several layers of air-shot mortar. The inner surfaces of the sidewall forms were placed 1 in. in from the final outer wall surface. The first layer of vertical and horizontal steel was fastened to the inside of the forms and covered with about 1 in. of mortar. The second layer of horizontal bars was then put in place and covered with another coating of mortar. Two unreinforced coatings, 1½ in. and ¾ in., were shot in place making the total thickness 5 in. The interior was finished with a ¼ in. flash coat. After the forms around the outside were removed a light wire mesh was placed over the entire outer periphery and a final 1 in. seal coat applied. Reinforcing for the roof consisted of a layer in an 8 ft. ring around the outer edge. A ring 4 ft. wide was shot first around the outer edge and the remainder of the roof shot the following day. Sand pockets were prevented by washing the exposed surfaces with compressed air and water before additional coats were shot.

The fair for new building materials

E. Probst, *Beton und Eisen*, Vol. 37, No. 14, p. 230, July 20, 1938 Reviewed by A. U. Theuer

Dr. Probst describes new materials on exhibit at the recent Leipzig Fair (Germany) which may be of interest to the concrete constructor. Among those intended for use as protective coatings on exterior surfaces, Decelith, a polimerized vinyl resin product (Vinglpolimerisate), is singled out for special attention. It is described as a tough, water resistant, non-corrosive and fire-proof material. Marketed in sheets

from 0.2 to 35 mm. in thickness, in bars and in special shapes, it is said to be unaffected by acids (excepting nitric) and alkalies, insoluble in organic solvents and oils, and to be water repellent. It has a modulus of elasticity of 570,000 p.s.i. and is stated to serve as a substitute for lead foil. Of interest for interior wall treatments is a commercial acoustical glass product, Mangaroid. This material is also stated to have excellent heat insulating properties. A number of special concrete form oils are mentioned. Among the concrete exhibits nothing particularly new is reported. These exhibits are stated to present innovations to present practice that serve only to emphasize the wide field and great adaptability of concrete for structural purposes.

Reactions between silica and calcium oxide in aqueous solutions and their relation to pozzolanic action

ALFRED E. BEITLICH, *Journal of the American Chemical Society*, Vol. 60, No. 8, pp 1832-1836, 1938. Reviewed by ROY N. YOUNG

The reactions of CaO dissolved in water with SiO_2 (silicic acid) were studied by means of an electrical conductivity method which proved to be very convenient and accurate. It was definitely established that, under the condition of the experiment, adsorption reactions take place. Also compounds are formed having molecular ratios very close to 1:2, 4:5, and 2:1, corresponding to $CaO.2SiO_2.xH_2O$, $4CaO.5SiO_2.xH_2O$, and $2CaO.SiO_2.xH_2O$. In dilute concentrations CaO is adsorbed by the SiO_2. With initial concentration ratios of CaO/SiO_2 between 0.30 and 0.60, the 1:2 ratio-compound is formed. As the CaO concentration is further increased adsorption again takes place until the ratio reaches 0.90. From 0.9 to 1.3, the reaction product has a $CaO:SiO_2$ ratio of 4:5. In the concentration range 1:3 to 3.75, adsorption reactions take place. In the saturated solution the CaO/SiO_2 ratio of the reaction product approaches 2:1, indicating a hydrated $2CaO.SiO_2$.

Recent developments in lightweight, burned-clay for concrete aggregates—gravelite

ROBERT D. PIKE, *Rock Products*, Vol. 41, No. 8, Aug. 1938, pp. 71-72. Reviewed by ROY N. YOUNG

Gravelite, a lightweight concrete aggregate can be produced from most clays and shales which are used in making common brick. The coarse aggregate is made by pugging the ground raw material, forcing the mixture through dies and then expansively burning this product in a rotary kiln. The size gradation is controlled either by using a die containing numerous holes of various sizes, or by crushing the burned material, screening and recombining.

The sand is made by grinding the raw material to an extent to give the desired gradation of the finished product—fineness modulus 3.25 to 3.5.

Two typical concrete mixtures and their properties are given. The weights of the wet concrete per cubic foot were 103 and 107.5 pounds, depending on the maximum size of the aggregate.

Gravelite concrete has considerable thermal insulation value, $K = 2.68$, compared with 10 to 12 for normal concrete.

Corrugated roads for safety

Highway Builder Magazine, Vol. 17, No. 9, Sept. 1938, p. 11 HIGHWAY RESEARCH ABSTRACTS

A highway geared to automobile tire treads has been constructed as an experiment by the Pennsylvania Department of Highways. The corrugated stretch, 3,500 feet long, is now serving traffic on the Lakes-to-Sea Highway, U. S. 322. Corrugations,

not unlike the imprints of a heavy tire, were molded into the wet concrete by vacuum mats during construction. On the experimental sections, the new surface, three inches thick, was laid without reinforcement over the old concrete road which had deteriorated under the pounding of an average daily traffic of 4,000 vehicles.

Advantages expected from the new process include further protection against skidding in all weather conditions, and reduction of sun and wet glare. Tires passing over the corrugations produce a humming sound but the treads are set closely together to prevent rough riding.

Briefly, here is how this type of road is constructed. The old surface is chipped down to the solid concrete, swept clean and washed. A cement paste, mixed to a consistency of thick cream, is smoothed over the washed surface. Vacuum mats then are used to remove excess water and imprint the corrugations.

Effect of glass content upon the heat of hydration of portland cement

WILLIAM LERCH, *Research Paper RP1127*—National Bureau of Standards, Vol. 21, No. 2, Aug. 1938.
AUTHOR'S ABSTRACT

The heat of hydration was determined, by the heat-of-solution method, for a number of portland cements prepared from clinker which had been subjected to various heat treatments to produce different glass contents. The results indicate that, when different cements are ground to about the same degree of fineness, the quantity of heat evolved between 7 and 28 days will be dependent on a number of factors, two of the most important of which are: (1) The composition of the cement, and (2) the glass content of the clinker. The glass content not only influences the heat of hydration directly, but also influences it indirectly, through its effect upon the true compound composition. No consistent relation was observed between the glass content and the quantity of heat evolved at 3 days. At 7 and 28 days the quantity of heat evolved for any given composition increased consistently with increasing glass contents. Some reasons are advanced to explain why the difference in glass content, together with the changed compound composition resulting therefrom, may effect the heat of hydration of the cement at various ages.

Properties of portland cement and methods of determining them

GABRIEL A. ASHKENAZI, *Cement and Lime Manufacture*, Vol. XI, No. 6, June 1938, pp 119-124.
Reviewed by J. C. PEARSON

The writer presents a mildly critical review of the newer developments in cement testing, particularly with reference to deductions that have been made regarding heat of hydration, compound composition, and the correlation of physical properties with composition. He is for specifying heat of hydration, not in calories per gram but in strength-heat ratios. He prefers the electrical method for setting time to mechanical or thermal methods. He believes the metalographic method for the study of clinker is more promising that the study of thin sections, and is not yet convinced that clinker is simply a mixture of four major compounds and a certain amount of glass. Purely chemical methods have not resulted in any essential development, but favorable reference is made to the perchloric acid method for silica, and to the various methods for determining free lime. With reference to relations between composition and physical qualities of cement, the author's opinion is that strict general limitations on certain of the oxides are not justified in the present state of our knowledge.

Fuel efficiency in cement manufacture, 1909-1935

NICHOLAS YAWORSKI, VIVIAN SPENCER, GEOFFREY A. SAEGER and O. E. KIESLING, Works Progress Administration National Research Project and Department of the Interior, Bureau of Mines.

The report, one of a number made by the Works Progress Administration, on the use of fuel saving devices and substitutes for coal in major industries which have brought about increasing unemployment in the coal fields, points out that the average consumption of energy at American cement plants declined from 189 pounds of coal or coal equivalent in 1914 to 159 pounds in 1935. Continued progress in fuel efficiency in the cement industry is expected as new inprovements are adopted and as equipment already successfully applied is more extensively introduced. "The rate at which further technical gains will be won, however, will be governed by the trend of such economic factors as fuel prices and cement production, which determine the money value of the savings and whether they will pay for additional equipment," the report adds.

The report is bound in multilith form and is of 92 pages, with numerous charts and tables and a complete statistical index. Requests for copies of the report should be made to the Publications Section, Information Service, Works Progress Administration, Washington, D. C.

Reinforcing steel for concrete quality-lengths, and impact-shock

DR. ING. BUEHLER, *Beton und Eisen*, Vol. 37, No. 16, p. 258 (Aug. 20, 1938). Reviewed by A. U. THEUER

In certain cases it is shown that cold rolled steel is to be favored because bars may be obtained in lengths of 160 to 200 ft. Furthermore, the production of a satisfactory product from the metallurgical standpoint and one that possesses the best cold working properties favors its use. The problem of transporting bars having lengths in excess of 160 ft. is stated to offer no great difficulty (in Germany). Precautions to be taken in making shipments by rail or highway are described in detail.

The author then takes up the subject of tensile shock and describes a number of tests made to determine the effectiveness of bar connections. The fatigue limit of stress connections, as shown by his tests, were found to be extremely low. Threaded couplings were found to require a cross-section three times as large as that of the bars connected. Difficulties with ordinary welded joints are described. In the case of resistance-welds more favorable conditions are expected even though no experimental data are at hand. The author concludes with the recommendation that at this time tensile shock in bars subjected to high alternating stresses should be avoided and that the use of continuous bars alone offers a satisfactory solution.

Notes relating to the physical tests of cements

EDITH COUILLAUD, *LeGenie Civil*, Vol. CXII, No. 16, Apr. 16, 1938, p. 337 Reviewed by R. L. BERTIN

An intensive study was conducted in nine laboratories, official and private, for the purpose of improving the homogeneity and reliability of compressive tests of cement mortars.

It was found that despite the greatest care in following a precise technique in all details manually, the mean results varied from 4 per cent to 15 per cent from one laboratory to another, and that the human element was the preponderant cause of the variation.

To eliminate the human factor, mechanical means of preparing the specimens were developed in which the mortar is prepared in a small rotary mixer yielding a uniform

mixture, fed slowly into a mold to which is imparted a horizontal eccentric rotation.

Results of tests show that with this new technique, the density of the specimen was increased about 4 per cent, the variation of compressive strength between laboratories was reduced from an average of 5 per cent to 7 per cent to a value below 2 per cent and that the spread above and below the average was reduced in the ratio of 3 to 1.

Additional tests are being made to evaluate the influence of the variables, such as time of mixing, filling, speed and eccentricity of mould rotation, etc.

Use of high alumina cements in the manufacture of refractory concretes

E. L., *LeGenie Civil*, CXIII, No. 1, July 2, 1938, p. 15-17 Reviewed by R. L. Bertin

Refractory concretes are those in which (1) the hydraulic binder is high alumina cement and (2) the aggregate is made from crushed silico alumina bricks or the like, calibrated granolometrically the same as the usual sand and gravel used for ordinary concrete.

The article gives the results in the form of graphs of tests conducted to show the effect of temperature on the compressive resistance of mortars compared to high alumina cement and aggregates less than 3 mm. in size, also of neat high alumina cement pastes at the age of 3 and 7 days. Other tests are reported showing the relation of expansion and contraction to temperature from 0° to 1400° C under a constant pressure of 1.8 kg/cm². These tests are in close agreement with similar tests conducted at the Mellon Institute in which the specimens expand up to a temperature of approximately 1200° C when they gradually contract until failure at about 1400° C.

The article is taken from a paper by Vivian Hussey "Aluminous Cement As A Bond For Refractory Concretes," published in *Chemistry and Industry*, Jan. 16, 1937, p. 53-61.

Verbund concrete

K. Krah, *Verkehrstechnik*, 1938, 18 (5), 129-30, Road Abstracts, London, June, 1938, No. 161.
Highway Research Abstracts

The "Verbundbeton" (Compound Concrete) method of construction consists essentially in the provision of a base course of dry concrete entirely enclosed by waterproof bituminous material and supporting a light bituminous surfacing. The foundation may consist either of an existing surfacing or of ordinary consolidated soil; accurate levelling is unnecessary. Old surfacings are covered with a priming coat of tar which is also applied to the vertical faces of the kerbs; stone chippings are then spread over the tarred base. Soil foundations are covered with building paper, which is bent upwards at the margins in order to protect the vertical faces of the concrete. The water-cement ratio of the concrete should be kept low; the suggested maximum of 0.4 includes the water content of the emulsion used in the later stages of the work. The concrete is placed, tamped, and covered with coarse chippings or broken stone, followed by a penetration dressing of weak bitumen emulsion. The water in the latter is rapidly absorbed by the dry concrete, the particles of bitumen remaining in the pores, while only a thin film of bituminous material remains on the surface. The concrete is again tamped and a wearing course is applied; this consists of a single or double surface treatment or of a thin course of asphaltic or bituminous concrete.

High concrete bridge for low cost

E. A. WHITE, *Engineering News-Record*, Vol. 121, No. 9, Sept. 1, 1938, pp. 265-267
Reviewed by S. J. CHAMBERLIN

The use of long columns and of hollow-type construction in all vertical and horizontal members resulted in constructing the Gehring Road Bridge near Tacoma, Washington for $84 per foot. Two hollow girders 4 ft. wide spaced 7 ft. apart carry the 20 ft. roadway slab. Except for hollow crossbeams connecting the girders at the bents, the deck construction and deck formwork is continuous, from one end of the bridge to the other. The girders are 5 ft. in depth excepting at the ends of the 90 ft. central span where they are deepened by means of flat haunches. The bents are hollow columns, 4 ft. square with permanently embedded, 3 ft. octagonal forms. At one end of the bridge the short bent was constructed monolithic with the deck. At the other end rollers consisting of extra-heavy 8 in. pipe filled with concrete were provided between the girders and the continuous deck. The center bents are tall enough to deflect slightly. The interior core forms, built of shiplap supported by 2 x 4 in. wood frames, were wet down before concreting so that future rewetting would not cause an expansion detrimental to the hardened concrete. The tall columns were placed in 8 ft. lifts and the concrete vibrated. The concrete in the bottom of the girders beneath the core forms was placed into the web of one side only until the concrete started to rise in the opposite web.

Physical and mechanical effects of trass and blast furnace slag on mortar and concrete

R. RODT. *Tonindustrie Zeitung*, V. 62, No. 41, 42, 44, 45/46, p. 461-3, 471-2, 490-2, 502-3, May 23, 26, June 2, 9, 1938.
Reviewed by A. E. BEITLICH

The plasticity of a portland cement mortar is improved when 25 to 30 per cent of the weight of the cement is replaced by trass. With blast furnace slag cement, the replacement of cement by trass should not be greater than 10 per cent. The elasticity of mortar or concrete containing trass is far greater than that of mixes without trass. Increases by at least 20 per cent of the elasticity are possible. With blast furnace slag cements, the elasticity is not greatly different from that of portland cement. Trass increases the shrinkage especially with coarsely ground cement. The heat of hardening is lowered considerably by replacing part of the cement by either trass or blast furnace slag. Concrete containing slag cement gives higher strength than portland cement concrete in water storage. The slag reacts chemically and aids in the development of strength. Reinforcements do not rust in slag concrete when properly placed. All mortars with or without trass show a steady increase in strength during a 2 year period. When the trass is added to the cement, the strength is higher, when the trass replaces the cement, the strength is lower than that of the plain cement. Trass improves the impermeability of the concrete and thereby its resistance against attack.

Contribution to the constitution of portland cement

K. KOYANAGI, S. KATOH and T. SUDOH, *Zement*, Vol. 27, No. 24, June 16, 1938, pp. 363-367
Reviewed by L. T. BROWNMILLER

An experimental cement was prepared from pure chemicals with an alumina content of 10.57 per cent. After grinding the material passing the 10,000 mesh sieve was separated into seven fractions by centrifuging. The fractions of lower specific gravity showed the higher Al_2O_3 and CaO and the lower Fe_2O_3 add SiO_2. The x-ray diffraction patterns showed the lines of tricalcium silicate and tricalcium aluminate in all fractions. According to mathmatical calculation the heaviest fraction should

not have contained any tricalcium silicate. The pattern of tricalcium aluminate in the cement showed a contraction of the lattice over that of the pure compound.

A series of melts were then made in the systems $C_3A - C_4AF$ and $C_3A - C_3S$. Tricalcium aluminate may take up tetracalcium alumino ferrite in solid solution with an expansion of the lattice. In the system $C_3A - C_3S$ a contraction of the lattice of tricalcium aluminate occurred similar to that observed in the cement composition. The conclusions drawn are: (a) in cements high in alumina the tricalcium aluminate takes up some tricalcium silicate in solid solution; (b) in cements of medium alumina the tricalcium silicate probably takes up some tricalcium aluminate in solid solution.

The evidence the authors present is almost entirely x-ray data with but little confirmatory data of optical or thermal nature; accordingly it might be well to supplement their data particularly with observations on the effect of the rate of cooling on the diffraction patterns.

The determination of slag in cement

F. Keil and F. Gille, *Zement*, Vol. 27, No. 36, Sept. 8, 1938, p. 541-546.
Reviewed by L. T. Brownmiller

Special cements are on the market which contain blast furnace slag ground with cement clinker. This article outlines a procedure for the determination of the relative amounts of the two materials in such cements. For the determination the following three properties of the cement and slag may be used as a means for their differentiation: (a) the specific gravity; (b) the chemical composition; (c) the microscopic properties.

On the basis of the variation of these properties in the cement and slag, the following methods may be used for the estimation of the relative amounts of the materials in a mixture: (1) separation with liquids of varying specific gravity; (2) chemical analysis; (3) planimetric analysis. Except in special cases it is necessary to follow through two or three of the procedures in order to get results which are significant.

In carrying out the methods of liquid separation or of planimetric analysis, the finer portions of the cement must be discarded. This may introduce a considerable error in the result since the ratio of slag to clinker may vary in the portions of various size ranges.

The authors give a detailed example of their method of analysis in which they proceed without a knowledge of the chemical composition of either the cement or slag. Their problem is a difficult one. In the article they have brought out many points of importance in carrying out such an analysis. In spite of all of the precautions they have given, the technique is laborious and not without chance, it would seem, of considerable error.

Temperature and stress in concrete beams and road slabs

Karl Eberle, *Die Betonstrasse*, V. 13, No. 7, pp. 149-156, July 1938. Reviewed by F. L. Ehasz

"On stresses of road slabs caused by temperature difference between the upper and lower surfaces and by loading" is the third and closing article of this series. For calculating stresses resulting from unequal temperature distribution in concrete slabs on an elastic foundation it is assumed, as in Westergaard's pioneer work, that (1) the slab acts as a homogeneous, isotropic, elastic solid, (2) the subgrade reactions are vertical only and proportional to the deflections, (3) the modulus of subgrade reaction is constant throughout, and (4) the fall in temperature in the slab is characterized by a straight line. Tests made at München and Stuttgart showed some interesting results and corroborated in large measure the Westergaard relations for stresses and deformations. Epitomizing the author noted that: (1) the modulus of subgrade reaction, which is largely a function of the soil's properties, appeared

to increase with time because of the condensation of the subsoil and to diminish with moisture penetration; (2) concrete on German motor roads at Stuttgart showed an average modulus of elasticity of 5,000,000 p.s.i. (350,000 kg/cm²); (3) Poisson's ratio may be taken as 0.15 (agreeing with test results of A. N. Johnson); (4) in consequence of actual prevailing temperature differences in slabs, according to calculations, significant stresses may arise on the lower surface; (5) temperature stresses can reach a multiple value of those due to the allowable load; and (6) the 8-2/3 in. (22 cm.) thickness stipulated in 1937 suffices according to calculations for the assimilation of stresses resulting from uniform temperature drop and allowable static loads. Weather-resisting aggregates having lower heat expansion and higher heat conductivity should be required.

Prevention of stain, streak and crack formation in concrete stone

LEO JOHN, *Betonstein-Zeitung*, V. 4, No. 8, pp. 223-225, Aug. 10, 1938. Reviewed by F. L. EHASZ

In this paper the author attempts to indicate the sources of difficulty encountered in work with colored concrete stone. He does not pretend to have a complete knowledge of the remedies of faulty construction, but rather indicates the importance of learning more about them through actual experience with a view to improving future endeavors. Care must be exercised in the choice of the raw materials, since impure admixtures are nearly always a cause of stains. Bad and incomplete mixing should be guarded against and it is to be emphasized that all the materials must not be mixed together simultaneously if stains and streaks are to be avoided. Proportions are to be measured according to weight and the proper sequence of mixing must be observed.

First, the coloring is most advantageously distributed and combined with the cement by mechanical means, for example, in a ball mill. A test for determining the efficacy of this operation is described in detail. While this process is under way the aggregate can be saturated with water to ensure proper union with the colored cement. The extent of remoistening of the final mix will be governed by the workability desired. Attainment of a completely uniform, dense surface is of paramount importance. In ramming the possibility of streak formation must be kept clearly in mind. Lack of consideration of richness and fineness of mix may affect shrinkage and manifestly induce ugly crack formation for the prevention of which the correct grain combination and provision for expansion are essential. The author's concluding remarks pertain to the need for cooperation between the artist and concrete technician in artistic work.

Blast furnace slag cement research

RICHARD GRUN, *Tonindustrie Zeitung*, V. 62, No. 66, 67, 68, p. 716-8, 729-31, 743-4, Aug. 18, 22, 25, 1938
Reviewed by A. E. BEITLICH

A report of the activities of the Research Institute of the Blast Furnace Slag Cement Industrie in Germany for the year 1937. Little change occurred in the chemical composition of the slags. The clinkers used are normal portland cement clinkers, some of which show a tendency towards higher Fe_2O_3 contents. The initial setting time of the cements is 4 to 5 hrs. The residue on the 5000-mesh sieve (approx. 178 mesh per sq. in.) is always less than 5 per cent. Little difference in strength exists between normal portland and blast furnace slag cement, the initial strength of the former being generally higher and the final strength of the latter being higher. The ammonium chloride method for silica determinations has been used very successfully. Efforts were made to replace the manual filling of bending test molds by mechanical devices. Considerable irregularities of the strength development were

encountered in a study of the curing method of specimens. No relation was found between the 28-day shrinkages of mortar and concrete. In studying the effect of slag additions on the shrinkage, it was observed that with an increasing slag content, above 60 per cent the shrinkage was generally less. The slag in the cement favors the development of strength at later ages and such cements are distinguished by their high strength and elasticity. Additions of gypsum to slag cements resulted in quite different strength developments. While some of these slag-gypsum cements hardened excellently, others were inferior, especialy their initial strength. The best results were obtained with cements containing 30 per cent clinker and 3 per cent gypsum. Cements with a low clinker content show low heat development on setting and are therefore recommended for mass concrete work.

Properties of asbestos-cement sheets

ANNONYMOUS, *Cement and Lime Manufacture*, Vol. XI, No. 8, Aug. 1938, p. 182.
Reviewed by J. C. PEARSON

A three-year study of specification tests suitable for asbestos-cement roofing materials is reported in the report of the Building Research Board for 1937. Accelerated weathering tests were made on the effects of wetting and drying, freezing and thawing, acid rainwater, and atmospheric carbonation. In the wetting and drying tests (immersion in water at 20° C. and drying at 50° C.) 480 24-hour cycles were completed. The effect was generally favorable in that moisture movement and absorption were reduced, while strength and modulus of elasticity increased slightly. Samples subjected to 250 cycles of freezing and thawing (18 hours at 15° C. and 6 hours in water at room temperature) showed lamination and flaking, the effect being in general the greater the larger the absorption. Relative freezing and thawing resistance was found related to initial values of modulus of elasticity. Change in the latter, however, seemed to have no relation to the amount of deterioration produced. Absorption increased and modulus of elasticity decreased, with repeated freezing and thawing. Specimens subjected to running water containing dissolved CO_2 or SO_2 showed little or no difference in rate of attack for different materials. Effects of atmospheric carbonation were as follows: Transverse strength increased but impact strength decreased; moisture movement decreased but there was an overall shrinkage. This was partly offset by the formation of water from the reaction, the net effect being a shrinkage of about 0.1 per cent. Decrease in impact strength was regarded as the most serious, which in ten years of exposure has been observed to be of the order of 30 to 50 per cent. Permeability was small and not deemed essential for the specification tests, which as a result of this study tentatively include transverse strength, water absorption and resistance to solution by acid waters.

The driving of concrete piles

P. CAUFOURIER, *LeGenie Civil*, Vol. CXII, No. 26, June 25, 1938, pp. 540-543
Reviewed by R. L. BERTIN

The Building Research Board of England conducted a series of tests to determine the behavior of the piles during driving and establish the optimum conditions relative to the construction of the piles and that of the driving mechanism, such as weight and fall of the hammer, general construction of the head, etc. The results of the tests, reviewed in this article, are outlined in detail in the report of the Board dated April 27th, 1938. The research work done by the Board covers:

(1) The analytical and experimental study of the nature and importance of the strains produced in the pile by shock.
(2) The study of the relation between the construction of the pile and its resistance to shock.

(3) The development of methods indicating dangerous conditions during driving.

The major conclusions drawn from the investigations are:

(1) The rupture of a pile under driving is, in general, due to excessive compression.

(2) The compression at the head is in general greater than at any other point.

If the pile is longer than 9 meters it is independent of penetration.

The kind of cement has little effect on resistance. There is no advantage in mixtures richer than 1:1-1/2:3 with as low a water cement ratio as is consistent with proper placing.

The longitudinal reinforcement does not have much influence on the behavior of the pile during driving; it is of value in preventing cracks during handling. 1 per cent transverse reinforcement for a length from 2 to 3 meters from each end is essential. In driving, as heavy a hammer and as light and elastic a head as possible should be used. The drop should be such as to limit the compression to 50 per cent of its ultimate. The weight of the hammer should not be less than that of 9 meters of pile.

The surface of contact between the head and pile must be level and the elastic padding must be of such material as to maintain its resiliency.

Graphs furnish information regarding the compression resulting from various driving conditions, weight of hammer, stiffness of the head, etc., from which safe driving can be evaluated.

New investigation of the form of particles of concrete aggregate

R. FERET, *Annales de L'Institut technique du Batiment et des Travaux Publics*. Vol. 3, May-June, 1938. p. 31. Reviewed by W. H. HERSCHEL

Various methods which have been proposed to describe the shape of particles were described in a previous paper. (Reviewed Sept.-Oct. 1937, p. 84). These methods are too complicated and are inapplicable to fine aggregate. In the present investigation materials were used which will pass one sieve and be retained on the next, such a size being designated as $a/2a$, a being the diameter of openings in the finer sieve. The weight of the material contained in the volume of a^3 cm^3 was determined, and this value, multiplied by the number of particles per gram, gives the number of particles in the volume a^3, a number which, with particles of the same form, should be nearly independent of their size. It was found possible to separate out the "flats" by means of sieves with rectangular openings. For the sizes of aggregate mostly used (2/4, 5/10, and 10/20) these openings were 1 by 10, 2.5 by 20 and 5 by 30 mm. Elongated particles, defined as having a length greater than $3a$, could not be separated out mechanically, and were picked out by hand only for the 10/20 size. Two types of material were used in tests, one with angular and the other with rounded particles. It was found that for a given type and size of material, the compactness is greater for the part retained on the rectangular sieve (i. e. with the flats removed) and for size 10/20 the compactness is greater when the length of particles is less than 30 mm. than when the length is greater. Compactness is defined as the absolute volume of the material contained in unit apparent volume. Since the number of particles in the volume a^3 is greater for the flats than for material retained on a sieve with rectangular openings, although the compactness is lower, the number of particles in volume a^3 does not appear to be of significance.

Compression strength tests were made on 13 mixes of concrete in three series. First the concretes were made plastic so they could be handled readily, then they were made of maximum possible density by vibration, and in the third series the water content was kept constant. Considering mean compactness of the three sizes

of material as a possible criterion for the mix, it was found that with the plastic concretes the strength increased with the compactness, but concretes with elongated particles were noticeably weaker. With vibrated concretes the relation between strength and compactness was different for angular than for rounded materials. It was noted that the marked decrease in compactness and strength due to additions of flats, ran parallel to the increase in mixing water required to keep a constant consistency. The third series of tests, however, showed that the increase in mixing water was not the only cause for this decrease.

The best and simplest way to determine the proportion and study the effect of flats is by sieves with rectangular openings. It seems logical to prescribe limits to the proportion of flats and of elongated particles in a given size of aggregate.

English specifications for the use of high alumina cement in reinforced concrete construction

LeGenie Civil, Vol. CXII, No. 13, Mar. 26, 1938, pp 272-274 Reviewed by R. L. Bertin

High alumina cement was originated by Bied in France about 30 years ago. Many rules governing its use have been written, the most authoritative are those published by the Institution of Structural Engineers of London in December 1937, entitled "The Use of High Alumina Cement in Structural Engineering." A report for the guidance of structural engineers when using high alumina cement is reviewed in this article and compared with the French regulations with which they are in close agreement. The subject is gone into in great detail, dealing with the following general characteristics:

(a) slow set (b) high resistance at short periods (c) resistance to chemical attacks (d) relatively dark color.

Used with crushed refractory materials, concrete capable of resisting temperatures of the order of 1300°C can be produced.

It is stated that the compressive resistance of 1-3 mortars gaged with water equal to 12½ per cent of the weight of the solids follows closely that of ordinary concrete mixed in the proportion of 4:2:1, (about 300 kg per m³ of cement) with a cement ratio of 1.66 at the ages of 1, 2 and 7 days, namely 590 kg/cm² at 24 hours and 670 kg/cm² at 7 days.

Neat mixtures of high alumina cement and water should never be used. Mixtures of high alumina and portland cement for use in reinforced concrete work are not recommended. Such mixtures are used where fast setting cements are needed; the time of set can be regulated at will by varying the proportions of the two cements.

Almost any aggregate suitable for portland cement concrete can be used with high alumina cement. The fines, particularly if of granite, feldspath mica or crushed schist are to be avoided.

The mixtures are about the same as for portland cement. However, sufficient aggregate should be present to carry water to the extent of 40 per cent of the weight of cement, the amount necessary properly to hydrate the cement. In general, mixtures containing 300 to 400 kg/m³ of cement and a cement water ratio of 2.5 to 1.7 are recommended as the range.

High alumina cement concrete should be mixed longer than portland cement concrete because of the harshness of the cement.

High alumina cement concrete, even more so than Portland cement concrete must be wetted as soon as the surface permits and for 24 hours; curing beyond that time is ineffective.

Form surfaces must be water soaked to prevent absorption and "burning." Stripping of forms can be effected between 12 and 24 hours after molding, based on a chart giving the per cent of 24 hour strength in terms of time.

The total heat liberated by high alumina cement is about the same as that of portland cement; however, it is released at a much higher rate particularly between the 5th and 12th hour after mixing. If the heat is dissipated slowly, the temperature may rise to a point injurious to the concrete. Hence masses thicker than 45 cm. should be avoided beyond which sprinkling becomes inefficient. This high heat, when properly controlled, is advantageous in cold weather.

High alumina cement is more resistant than portland cement to the corrosive action of neutral sulphates, water of zero hardness and weak acids. However, it is more sensitive to the action of concentrated alkalis.

The grading of aggregates and workability of concrete

W. H. GLANVILLE, A. R. COLLINS, D. R. MATTHEWS, *Road Research Technical Paper No. 5*, Department of Scientific and Industrial Research and Ministry of Transport Reviewed by T. C. POWERS

The authors first present data to re-affirm the relationship between strength and water-cement ratio and its independence of aggregate gradation. They find, however, that the water ratio relationship is valid only for fully compacted concrete. They find a definite relationship independent of water-cement ratio between a *strength ratio* defined as

$$\frac{\text{strength when incompletely compacted}}{\text{strength when completely compacted}}$$

and the *density ratio*, which is

$$\frac{\text{weight when incompletely compacted}}{\text{weight when completely compacted}}$$

For example, 5 per cent air void space reduced compressive strength 30 per cent; 25 per cent air voids reduced strength 90 per cent.

Workability is discussed theoretically, the authors taking the stand that the total useful work done in placing concrete is that required to overcome friction against the forms, reinforcement, etc. and that required to overcome internal friction, the latter being a characteristic of the mix. Following a discussion of the slump test, a test method is described which is based on the assumption that the amount of compaction produced by the application of a standard quantity of work is an index to the workability of the mix, that is, to the amount of work that would be required to produce complete compaction. This test is supplemented by the heap test and the slab test. The first "gives a photographic record of the shape taken by a mass of concrete when allowed to fall a given distance onto a flat surface, while the second. . . . . shows the surface texture obtained when the concrete is deposited in a standard manner into a mold of thin section. In this way the tendency of the concrete to segregate or to form honeycomb was recorded."

With the aid of these tests, factors affecting workability were studied. It was found that no ideal gradation existed but that "for any one value of water-cement ratio there was one proportion of sand to gravel that gave the greatest workability and, conversely, for one degree of workability there was one particular proportion of sand to gravel which needed the lowest water content."

The importance of gradation was found to be small in rich mixes but to increase "both with a reduction in cement content and demand for higher workability."

Rounded aggregates were found to require less water than angular ones.

The effect of rock dust on the water-cement ratio-strength relationship was found to be negligible. Limited amounts could be used without increasing the water-

cement ratio, the amount depending upon the degree of workability desired and, to a lesser extent, on aggregate gradation.

This is an unusually well written bulletin. The authors are to be complimented on their skillful presentation of the data and theoretical considerations.

It is to be noted that the conclusions regarding the relationship between gradation, workability, and water-cement ratio are practically identical with those which have previously been reported in this country based on other kinds of tests for measuring workability and on field experience. It does not appear, therefore, that the introduction of a new workability device throws any radically new light on the problem of workability.

However, this reviewer believes that the test device described in this paper for measuring the compacting factor deserves careful consideration. Theories aside, it is a fact that most concrete is placed under conditions somewhat comparable to those of the test; that is, a fixed and not a very large amount of work is applied, and the degree of compaction obtained depends to a considerable extent upon the characteristics of the mix. This is particularly true of the present methods of placing highway concrete. Perhaps a measure of the compaction factor of mixes as a standard procedure would lead to the production of more homogeneous pavement slabs through control of the workability factor, or, on the other hand, might show the advisability of regulating the amount of work expended in placing the concrete according to the actual requirements of the mix in use.

Determination of size gradation of aggregates, and practical information which it furnishes

J. Bolomey, *La Revue des Materiaux de Construction et de Travaux Publics*, No. 345, p. 106, June 1938.
Reviewed by W. H. Herschel

The more compact the aggregate, the less cement paste needed to fill the voids. The best grading of cement and aggregate for compactness and workability is defined by our equation,

$$P = A + (100 - A)\sqrt{d/D} \qquad (1)$$

where P is the weight of particles smaller than d expressed as per cent of weight of cement plus aggregate. D is the maximum diameter of particles. A may be taken from the following Table.

	Consistency		
	Damp (Vibrated concrete)	Pasty (Reinforced concrete)	Fluid (Poured concrete)
Gravel	A = 8	A = 10	A = 12
Broken stone	A = 10	A = 12	A = 14

The cement content per m³ of concrete corresponds in general to a certain per cent by weight of dry material, as follows:—

Cement, Kg/m^3	Cement, per cent of dry material
150	6.8
200	9.1
250	11.3
300	13.6

To obtain the grading of the aggregate alone, equation (1) becomes,

$$P_s = \left[A - C + (100 - A)\sqrt{d/D} \right] \frac{100}{100-C} \qquad (2)$$

or

$$P_s = (P - C)\,\frac{100}{100-C} \qquad (3)$$

where P_s is expressed as a per cent of the weight of aggregate and C is the cement content expressed as per cent of cement plus aggregate.

The theoretical grading may be approximated by mixing aggregates. In a certain case, for example, aggregate I contained 53 per cent of size 0-1.5, of which 21 per cent

was theoretically required, and aggregate III contained 75 per cent of size 15-30 mm. of which 30 per cent was required for the desired grading. Then 39 per cent of I and 40 per cent of III may be taken. If a 4-10 mm. gravel is used for the remaining 21 per cent of the mix, it would be deficient in size 0-0.3 mm and would be improved by taking 44 per cent of I, 16 per cent of II and 40 per cent of III. Then the deficiency in size 0-0.3 would be compensated for by an excess of 0.3-1.5 mm particles.

The amount of mixing water required may serve as a means for comparing aggregates. The weight of water e needed for a weight P of a component of the aggregate with size of particles between d_1 and d_2 is given by

$$e = \frac{P\,N}{\sqrt[3]{d_1\,d_2}} \tag{4}$$

e and P being expressed in kg or in per cent of the total weight of dry material. N is a coefficient which varies with the density of the stone and with the roughness and form of particles. For a density of 2.6 to 2.7, N may be taken as follows:—

Consistency	N for gravel	N for broken stone
Damp	0.080	0.095
Pasty	0.090 to 0.095	0.100 to 0.110
Fluid	0.100 to 0.110	0.120 to 0.130

For materials smaller than 0.2 mm, equation (4) is inapplicable, and e may be taken as 0.23 P for cement and river sand (0.1-0.35 mm) 0.35 P for powdered stone and 0.50 P for hydrate of lime. The total amount of mixing water, $E = \Sigma\, e$. If E is the same for two aggregates, the strengths of the concretes, with the same cement content, will be practically the same. It is thus permissible to deviate considerably from the theoretical grading, provided that E is not changed, and the content of fine sand is kept constant so as to preserve the workability.

The strength in compression may be estimated from

$$R = (C/E - 0.50)\,K \tag{5}$$

which is a simplification of

$$R = \frac{K/E}{\left[\, C/E \left(\frac{\Delta}{2.35}\right)^2 \,\right]^{3/2}}$$

where C/E is the cement-water ratio by weight, Δ is the density of the concrete and K depends upon the cement used, but may be taken as 220 kg/cm² at 28 days.

If for example R is to be 250 kg/cm², and by equation (4) the mixing water for the aggregate alone is 6 per cent of the aggregate S, then from equation (5), $C/E = 1.63 = \frac{C}{0.06\,S + 0.23\,C}$ from which $C = 0.157\,S$ and the cement will be 13.6 per cent of the weight of aggregate plus cement. It is thus easy to calculate in advance the strength of a concrete made from any aggregate, providing the grading of the aggregate, the cement content and consistency of the concrete and the characteristics of the cement are known. The amount of mixing water calculated as above indicated will rarely differ by more than 5 to 10 per cent from the mixing water actually needed to obtain a concrete of the desired consistency.

Investigation on the vibration of concrete: further tests on concrete compacted on a vibration table

F. G. THOMAS. *The Structural Engineer*, Apr. 1938, p. 142 Reviewed by N. H. ROY

The results of tests on vibrated concrete reviewed herein are reported in the Second Interim Report of the Joint Sub-committee of the Inst. of C. E. and Inst. of Structural Engineers in the April 1938 issue of *The Structural Engineer*. The first interim report appeared in the March 1937 issue of the same publication. A brief

summary of the conditions and conclusions of the preliminary tests covered by the first report is hereby given, followed by the results of the broader investigation.

The preliminary tests were made on 4-in. cubes of concrete of a 1:1.8:4.2 mix by weight of river sand and gravel, all aggregates passing the 31 in. screen. Specimens were vibrated by a machine having a range of 500 to 8000 vibrations per minute, amplitude from 0 to ⅛ in. and with accelerations up to 10 times gravity. The grading of the aggregates and the mix were held constant throughout the tests. The periods of vibration, length of time of vibrating, the magnitudes of the accelerations, the water-cement ratios were varied widely. The cubes were tested in compression at 7 days.

For the 4 in. cubes of the sand and gravel concrete used in the tests, the following conclusions are drawn.

(1) The use of vibration gives a satisfactory consolidation with a lower water-cement ratio than does hand tamped concrete, with a corresponding increase in strength.

(2) The acceleration should be above a certain critical value. For the mix used, 1 to 6 river sand and gravel, this value is about 4g for a water-cement ratio of 0.40, decreasing to 1.5g for a water-cement ratio of 0.60, when the time of vibration is 2 minutes.

(3) The frequency, vibrations per minute, is not of great importance. However, low frequencies for dry mixes, high frequencies for wet mixes, appear to be better.

(4) The time of vibration required to consolidate concrete, for a particular acceleration, is greater for higher frequencies, particularly for dry mixes. The increase in strength with time of vibration is relatively slow after a critical time is exceeded. This time, for an acceleration of 4g and the particular mix, ranged from about 5 minutes for a dry mix with a frequency of 8000, to 5 seconds for a wet mix, all frequencies.

If the time of vibration is greatly extended, the 7-day strength may be increased appreciably, particularly with wet mixes. As an example; with a water-cement ratio of 0.60, the strength for 3 hours of vibration was 70 per cent greater than for 2 minutes of vibration.

In further tests all specimens were cast on the large vibrating table. The frequency of vibration was 3000 per minute throughout. The same sand and gravel were used as in the preceding tests. However, various gradings were used in each of several mixes. The mixes were from 1:3 to 1:9, by weight. The time of vibration ranged from 15 seconds to 6 minutes with a constant acceleration of 4g, and from 1g to 10g with a constant time of 2 minutes. Studies were made of the effects of prolonged vibration and of re-vibration on the strength of concrete, 4 inch cubes.

Shrinkage effects on beams 36 by 4 by 4 in. for a 1:6 mix, 0.60 w-c ratio, (ages of 7 days to 6 months) were determined. The modulus of elasticity of vibrated concrete was determined from 4 x 10 in. cylinders at 7 and 28 days. Creep tests of 4 x 10 in. cylinders, loaded at the age of 3 days to a stress of 715 p.s.i. were also made. A series of bond strength specimens a ¾ inch steel bar embedded in a 4 in. cube; a series of modulus of rupture prisms, 4 x 4 x 16 in., and a few tests of the second interim report.

For the particular concrete used in the tests, and for the size and type of test specimens used, the following conclusions may be drawn, although such conclusions may require revision for large masses of concrete under different types of vibration.

(1) The grading of the aggregate does not affect the strength of vibrated concrete, with a particular water-cement ratio, provided the vibration is sufficient for satis-

factory consolidation of the concrete. The grading is of less importance for mixes richer than 1 to 6, by weight, than for leaner ones. A mix of 1:9 may be used, for some purposes, but requires careful consideration and tests to determine a suitable grading.

(2) For the mixes used in the tests, from 1:3 to 1:9, with the water-cement ratios ranging from 0.275 to 0.65, the relationship between the water-cement ratio and optimum 7-day compressive strength is linear. The optimum strength for 0.275 w-c is about 8800 p.s.i., and for 0.65 w-c ratio it is about 2600 p.s.i.

(3) The lowest possible water content for optimum strength is given for each of five mixes used in the tests as follows: 1:3 mix, 0.27; 1:4½, 0.33; 1:6, 0.40; 1:9, good grading 0.55.

(4) In some cases the use of a very rich mix with a given w-c ratio may give a lower strength than a leaner mix with the same w-c ratio. This is time when the water content is such that the use of the richer mix leads to a very wet concrete.

(5) Concrete of sufficient strength for many purposes, about 2500 p.s.i. may be obtained from a vibrated mix as lean as 1:9 with a comparatively low water-content, when a suitable grading is used.

(6) The effect of prolonged vibration of a wet mix is to throw out excess water and thus reduce the water-cement ratio and increase the strength. However a higher strength would be obtained by vibrating for a short time a mix whose water-cement ratio is equal to the value obtained with the wetter mix after prolonged vibration.

(7) Strength of concrete is but little affected by short period vibration subsequent to initial vibration for compacting.

(8) Shrinkage and creep are reduced by using lower water-contents and compacting by vibration; the modulus of elasticity and modulus of rupture are at the same time increased.

(9) Bond strength is somewhat higher for vibrated concrete with a lower water content than for hand-tamped higher water content concrete.

(10) The extensibility of concrete (ability to stretch without cracking) is apparently about the same as for hand-tamped concrete.

(11) It is thought the tests show that the use of vibration as a means of compacting concrete allows the use of a lower water-cement ratio than would be possible for hand-tamped concrete; and that the physical properties of vibrated concrete differ from those of hand-tamped concrete only in so far as they are affected by the change in water-cement ratio.

The Joint Sub-Committee and Dr. Thomas have contributed much valuable information to a better understanding of the effects of vibration on concrete mixes. Work is now in progress to develop a simple, compact, and inexpensive instrument for use in determining the characteristics of vibrated concrete on the job.

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Beams with Intermediate Expansion Hinges in Rigid-Frame Bridges*

By D. H. Pletta† and Leonard C. Hollister‡

MEMBERS OF AMERICAN CONCRETE INSTITUTE

SYNOPSIS

The use of multiple-span, rigid-frame bridges as crossings over major streams in place of truss spans has been due largely to the progress in construction and analysis of continuous structures. In the longer crossings, however, some provision must be made for expansion. This was originally provided for by discontinuity of the girders over an interior pier, a practice that destroyed the structural continuity at that point and adversely affected the aesthetics of the bridge. The California Highway Dept. finally overcame these difficulties by developing an expansion hinge at about the quarter point of an interior girder.

The incorporation of these intermediate expansion hinges in haunched beams complicated the design considerably and suitable charts were developed so that the hinged beams could be treated as similar solid beams in analyzing the structure by moment distribution.

This paper describes details of such expansion joints, their use in existing bridges, and presents a series of design charts which materially reduce the man-hours required for the analysis.

The evolution of highway bridge design during the last decade has been noted for its rapid change from the simple span to the continuous or rigid frame type.

The advantages of the rigid-frame type have become generally known and accepted. Methods of analysis have become more nearly standardized and made less burdensome by the development of curves for the elastic properties of beams. These factors have made it possible and natural to extend the scope of the rigid-frame type from the single and double span structure to the multiple-span bridge.

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Further, with the reduction of moments accompanying continuous frame rigidity and the increase in allowable working stresses due to improved construction methods, spans have been lengthened to such an extent that this type of structure can now be considered for bridges across major streams, where before the old type of simple-span concrete girders might have been excluded.

Some provision, however, must be made in the long multiple-span continuous structures for expansion and contraction. It is important that this provision preserve the advantages gained by maintaining the continuity of the design, and the aesthetics of the structure.

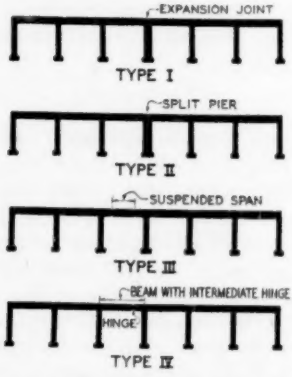


FIG. 1—FOUR TYPES OF EXPANSION JOINTS FOR CONTINUOUS-SPAN STRUCTURES

Fig. 1 shows four types of expansion joints which have been developed and used for continuous-span structures. Type I is a detail commonly used for simple-span girder designs. It consists of a pier with a cap enlarged to provide space for the two girder seats and their customary expansion details. This type obviously interrupts the continuity at the expansion pier, since the girders supported on this pier must have free or hinged ends resulting in an increase in design moments. It also interrupts the aesthetics of the structure, since the expansion pier is somewhat larger than would otherwise be necessary. Furthermore, this pier cannot be constructed as an integral part of the superstructure as is desirable for strength, rigidity, and appearance.

The "split column" in Type II interrupts the continuity of the design somewhat less than Type I, but it has practically the same effect on the adjoining girders as has freeing their ends. This type is difficult to construct properly and cannot be used for all conditions or in all types of structures.

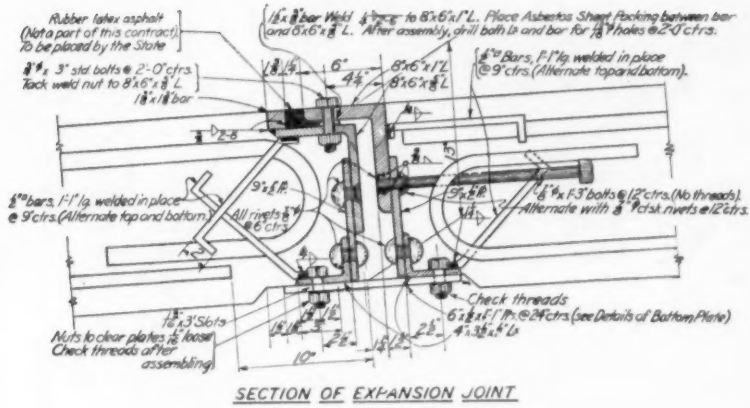


FIG. 2—TYPICAL DETAIL OF EXPANSION HINGE FOR SLAB TYPE OF STRUCTURE

The suspended-span arrangement shown in Type III is more desirable than either Types I or II. It interrupts the continuity of the design much less since the hinges can be placed near the points of contraflexure for dead loads. It does not interrupt the aesthetics; all piers can be identical, and integral with the superstructure.

A refinement in or improvement on Type III, however, can be made as in Type IV, by eliminating one of the hinges. The second hinge in Type III serves no real purpose, and it has the disadvantage of some interruption of continuity. The hinges are difficult and costly to construct, troublesome to maintain, and are a source of discoloration to the outer portion of the structure from the leakage of surface waters. It is therefore advisable to eliminate unnecessary hinges. The only advantage of the second hinge is in simplifying the mathematics of the analysis. It is the purpose of this paper to present curves for the elastic properties of beams with an intermediate hinge, the use of which will simplify the analysis, as has been done for other continuous beams and frames.

The Bridge Department of the State of California has developed and used this type of expansion detail on a number of multiple-span, rigid-frame structures during the last five years. It was first used on the San Gabriel River Bridge, (Fig. 9-E) built in 1933 at a cost of $109,000 or $2.30 per sq. ft.

Subsequent to the San Gabriel River Bridge construction, some thirty other structures have been designed making use of the intermediate hinge. Its use has been extended to the continuous slab

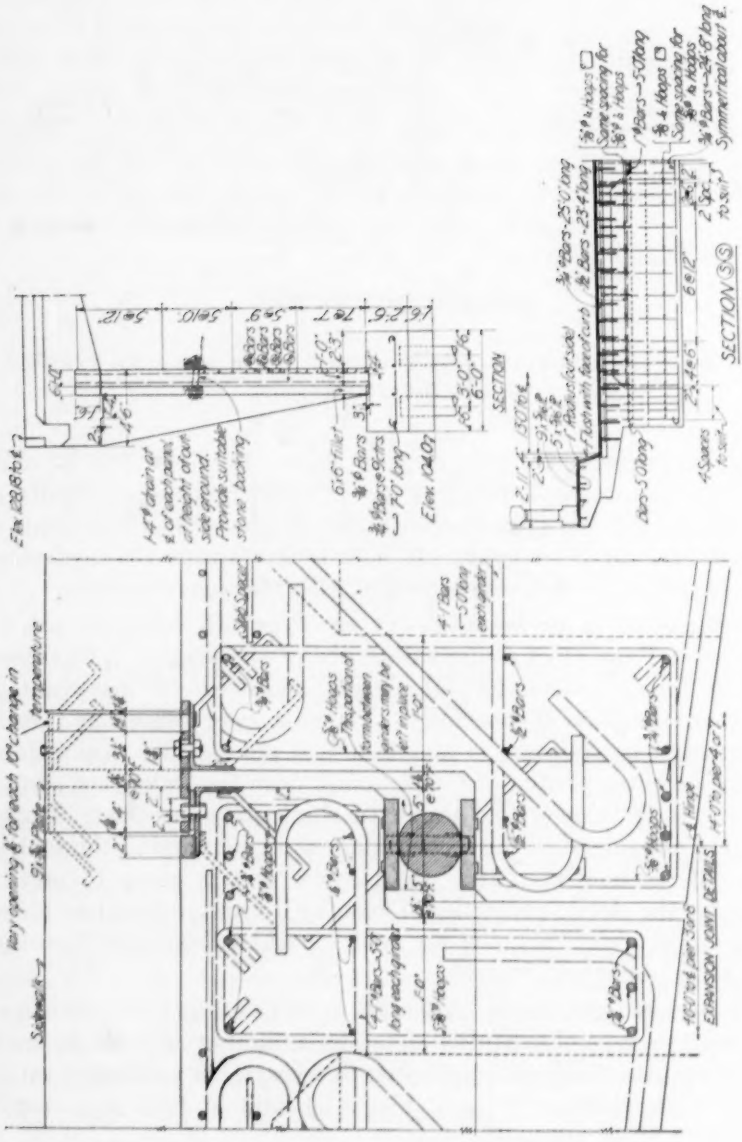


FIG 3.—SAN JOAQUIN RIVER BRIDGE—EXPANSION JOINT DETAILS AT LEFT

design, and, as can be seen by a study of Fig. 10, it takes care of the expansion problem for this type of structure most efficiently. A typical detail of an expansion hinge for the slab-type of structure is shown in Fig. 2.

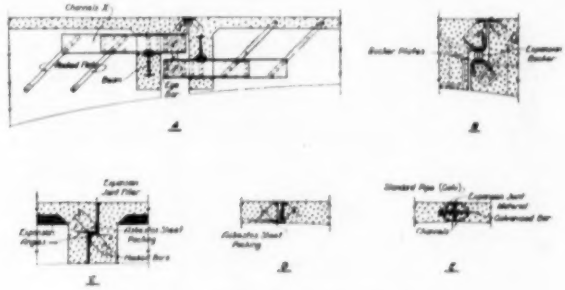


FIG. 4—HINGE DETAILS USED

Some study has been given to the proper location of the intermediate hinge. This has been found to lie at some point from 0.20 to 0.25 of the span-length out from the support. For beams with constant moment of inertia $\left(\frac{I_C}{I_A} = 1\right)$ the location should be about 0.20 L; for beams with a small $\left(\frac{I_C}{I_A}\right)$ ratio of moment of inertia at the center to moment of inertia at the ends, the distance (nL) out to the hinge should be about .25 L.

The spacing of the hinges throughout the structure, depends upon the stiffness of the bents or legs of the frames. In those structures with high, flexible legs it is possible to place the joints much farther apart than in those structures with short stiff legs. The maximum spacing of the hinges, in other words, depends upon the stress induced by the flexure of the legs of the frames due to movements caused by temperature changes.

Several types of details for the intermediate hinge to be used with the girder type of structure have been developed and used with success. Fig. No. 3 shows the details for the expansion hinge used on the San Joaquin River Bridge. It consists of an armoured joint at the top manufactured from angles and plates anchored to the concrete girders with ½ in. square anchor bars. The roller consists of a 5½ in. round structural steel pin at each girder. Heavy concrete cross-beams are placed between girders at the hinged joints to distribute the shears over an area greater than that represented by the width of the girders.

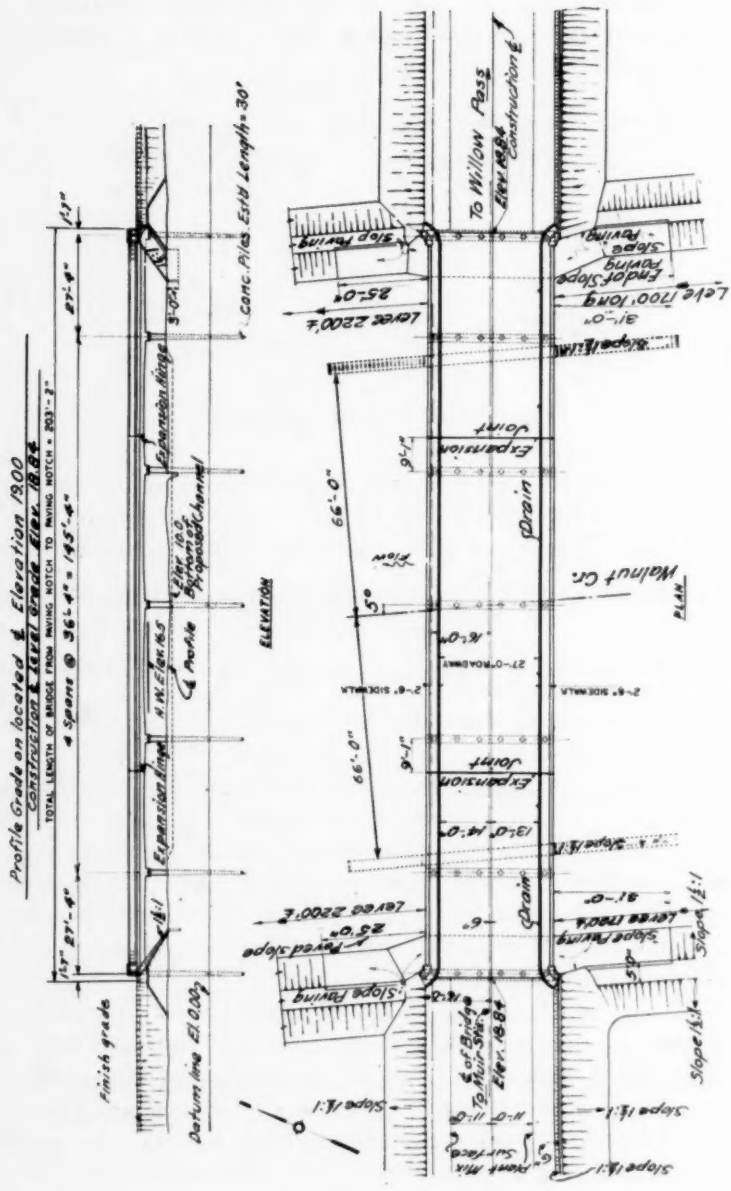


FIG. 5—FROM THE GENERAL PLAN AND LAYOUT FOR CACHE CREEK BRIDGE

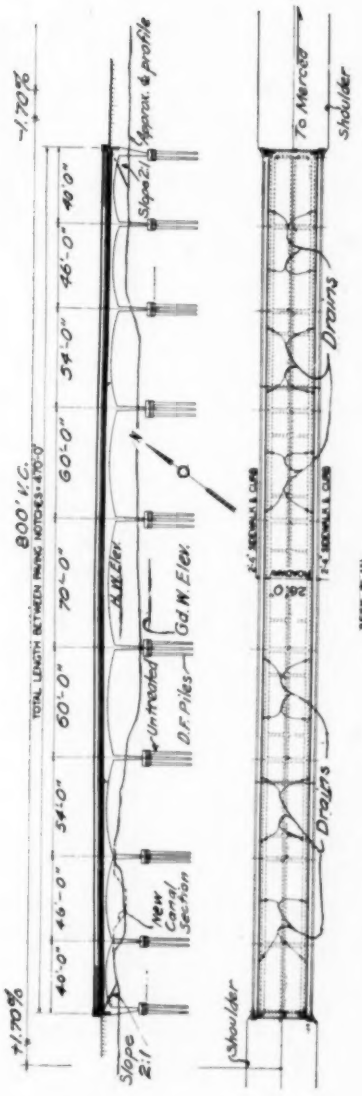


FIG. 6.—FROM THE GENERAL PLAN AND LAYOUT OF SAN JOAQUIN RIVER BRIDGE

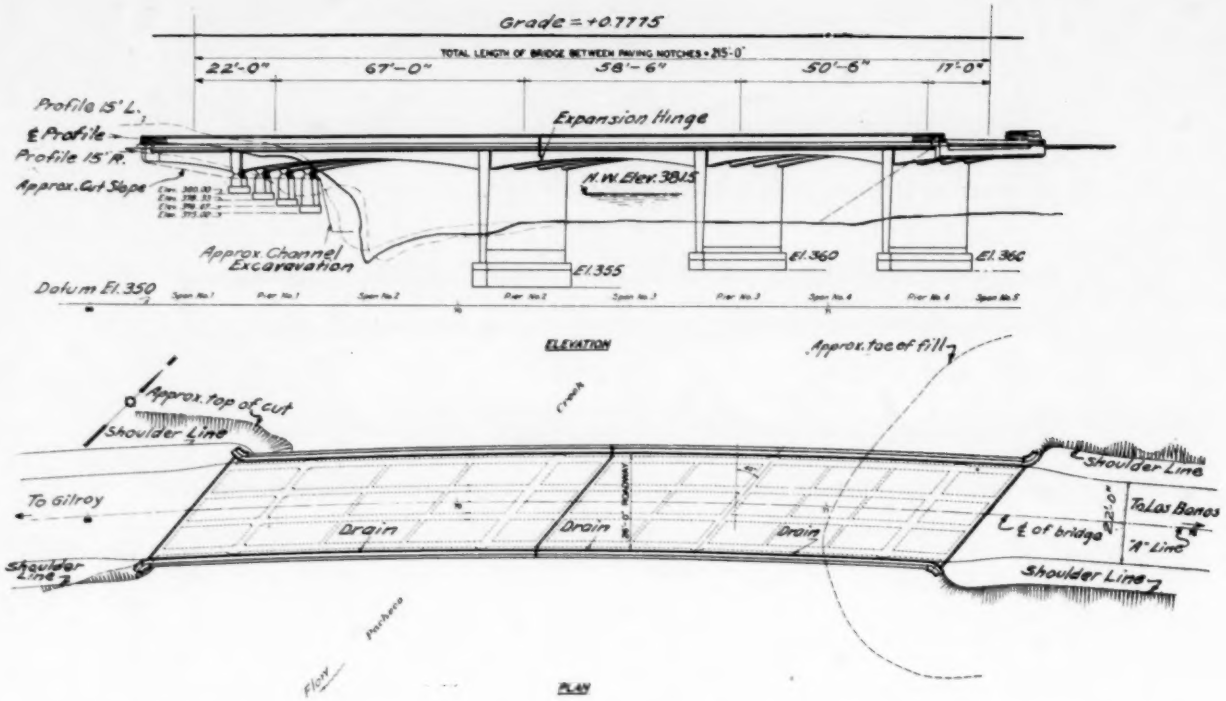


FIG. 7—FROM THE GENERAL PLAN AND LAYOUT FOR THE STRUCTURE ACROSS PACHECO CREEK

Fig. 4 (*A-B-C-D* and *E*) shows the general hinge details that have been used. At *A* is the type of hinge making use of structural steel shapes and eye-bar links, later found to be more elaborate and expensive than necessary. At *C* is the type used on the shorter girders and consists of two large angles providing a smooth sliding surface. The angles are separated by $\frac{1}{16}$ inch or $\frac{1}{8}$ inch asbestos sheet packing which has been found to give a very smooth, durable surface on which to slide. Type *D* is similar to the one shown in Fig. 2, except that it is supported at the bottom of the slab rather than at the top. Type *E* consists of 1 in. round steel rods on about 15-in. centers sliding in steel pipes. The shear is transmitted through the rods.

Fig. 5 shows the general plan and layout for a structure across Cache Creek. Expansion hinges in the second and fifth spans are 13 feet out from the pier. This structure is skewed 45 deg. and one end is on a 500-foot radius curve. The estimated cost of this structure is \$53,420.00—\$4.70 per sq. ft. of slab area—that is, "out-to-out" of roadway slab.

Fig. 6 shows the general plan and layout for a structure across the San Joaquin River. This structure has hinges in the third and seventh spans; and has span lengths decreasing from the main center-span of 70 ft. 0 in. The estimated cost of this bridge, now being constructed, is \$62,152.00 or \$4.32 per sq. ft. of slab, "out-to-out."

Fig. 7 shows the general plan and layout for a structure across Pacheco Creek. This structure is composed of 3 main spans 67 ft. 0 in., 58 ft. 6 in., and 50 ft. 6 in. with a 22-ft 0-in. cantilever at one end and a 17-ft. 0-in. cantilever at the other end. Of special interest is the fact that the girders of this structure are all built to the 2000 foot radius curve. In addition it is skewed about 40 deg. There is one hinge in the center span. The estimated cost (now under construction) is \$26,150 or \$4.05 per sq. ft, "out-to-out" of slab.

Fig. 8 shows the general plan and layout for Walnut Creek bridge, a typical slab type structure supported on reinforced concrete piles with expansion hinges in the second and fourth spans. The detail of the hinge is similar to that shown in Fig. 2. The estimated cost (now being constructed) is \$22,300 or \$3.19 per sq. ft. of slab, "out-to-out."

Fig. 9 (*A* to *I*) shows longitudinal sections of nine, multiple-frame girder-spans which make use of the intermediate hinge for the relief of expansion stresses. The structures and their estimated costs are as follows:

(A) Hayfork Creek Bridge	\$ 18,048.00—\$3.07 per sq. ft.
(B) Santa Clara River Bridge at Saticoy	105,075.00— 4.57 per sq. ft.

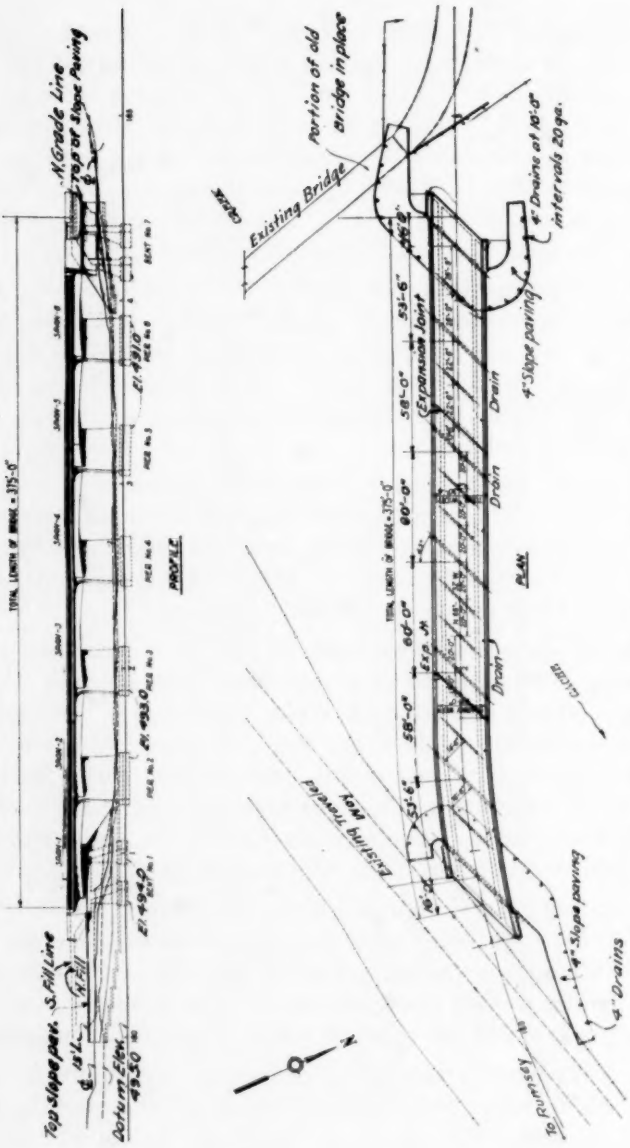


FIG. 8—WALNUT CREEK BRIDGE, A TYPICAL SLAB TYPE STRUCTURE SUPPORTED ON REINFORCED CONCRETE PILES

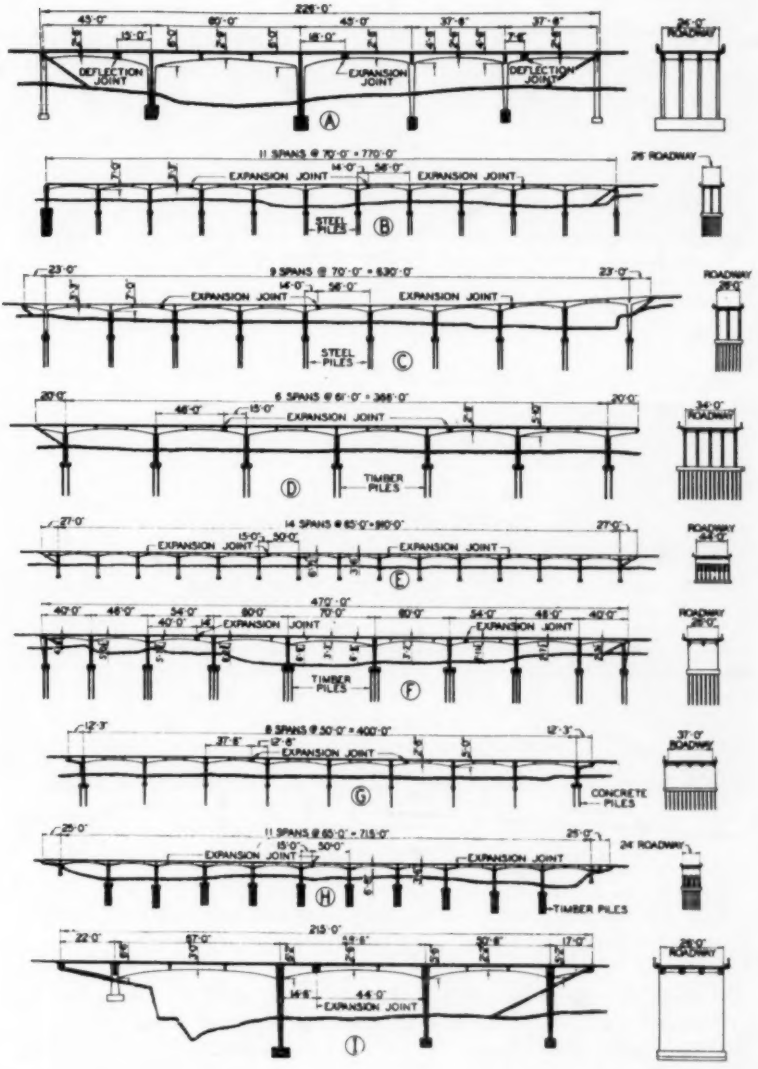


FIG. 9—LONGITUDINAL SECTIONS OF NINE MULTIPLE-FRAME GIRDER-SPANS USING THE INTERMEDIATE HINGE

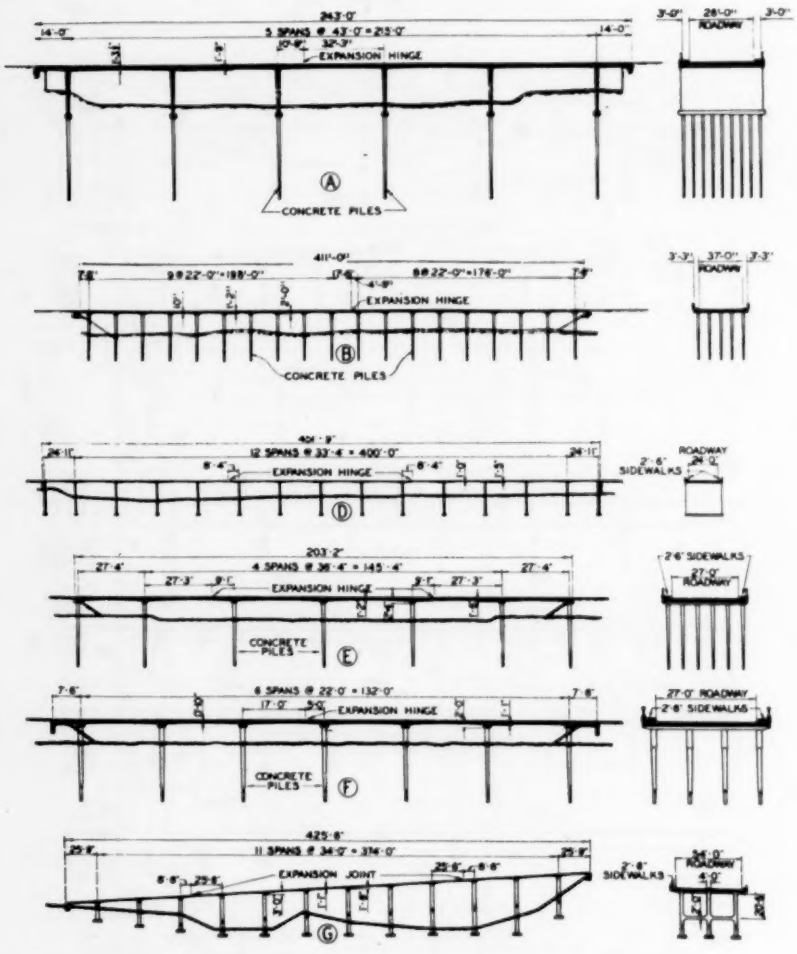


FIG. 10—LONGITUDINAL SECTIONS OF SIX CONTINUOUS SLAB BRIDGES USING THE INTERMEDIATE HINGE

(C) Santa Ana River Bridge at Norco	109,087.00	4.99 per sq. ft.
(D) Santa Ana River Bridge near Colton	76,485.00	4.70 per sq. ft.
(E) San Gabriel River Bridge	109,000.00	2.30 per sq. ft.
(F) San Joaquin River Bridge	62,152.00	4.32 per sq. ft.
(G) Santa Clara River Bridge	69,100.00	4.10 per sq. ft.
(H) Santa Ynez River Bridge	84,068.00	3.91 per sq. ft.
(I) Pacheco Creek Bridge	26,150.00	4.05 per sq. ft.

Fig. 10 shows the longitudinal sections of six continuous slab bridges making use of the intermediate expansion hinge. This type of structure has been found useful at those locations requiring the maximum

FIG. 11—LOS ANGELES RIVER BRIDGE ON ATLANTIC BOULEVARD
This structure consists of 13 spans of 64 feet and two 18-ft. cantilever end spans. The roadway width is 44 ft., with two 4-ft. sidewalks. The structure cost approximately $3.90 per sq. ft.

FIG. 12—HAYFORK CREEK BRIDGE
The longitudinal section of this structure is shown in Figure 9-A.

clearance, and where the hydrographic conditions permit the use of a short-span trestle of low cost. These structures and their costs are:

(A) Tule River Bridge	$37,520.00—	4.32 per sq. ft.
(B) Sweetwater River Bridge	24,782.00—	2.30 per sq. ft.
(D) Sespe Overflow Bridge	44,150.00—	3.08 per sq. ft.
(E) Walnut Creek Bridge	22,300.00—	3.19 per sq. ft.
(F) Grayson Creek Overflow	10,895.00—	2.18 per sq. ft.
(G) Pinole Overhead Structure	65,630.00—	2.85 per sq. ft.

The direct reading charts presented herewith were developed for hinged beams with a symmetrical parabolic haunch, the type most commonly used in practice. For other beam shapes, transformation equations are included in this paper so that existing charts for solid

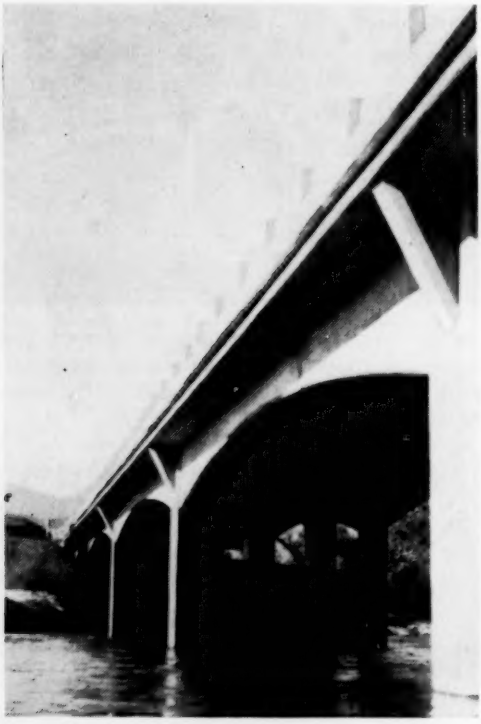


FIG. 13—KERN RIVER BRIDGE AT ISABELLA

Consists of 3 spans of 60 feet and 2 spans of 50 feet. There are 2 hinges (second and fourth spans). The structure has a 24 foot roadway.

beams may be utilized. The charts and equations enable the designer to obtain stiffness and carry-over factors and fixed-end moments for a variety of hinged beams and load conditions. He may then apply the method of moment distribution in the usual way to the structure as a whole and avoid complicating the analysis.

The charts are shown in Fig. 16 to 23 inclusive. It should be noticed in Fig. 16 that the carry-over factor is entirely independent of the beam shape, and affected only by the hinge location.[1] Hence the carry-over factor from end A to B is c/n. Fig. 17 shows fixed-end moments for a uniformly distributed load over the entire span. Fig. 18 to 23 give values for end moments due to a concentrated load on beams with hinges at five points along the span. Interpolation for

[1]For a proof of this statement and for the development of the charts and transformation equations see "Intermediate Expansion Hinges in Multiple-Span Rigid-Frame Bridges" by D. H. Pletta, a thesis submitted in partial fulfillment of the requirements for the Degree of C. E. at the University of Illinois, 1938.

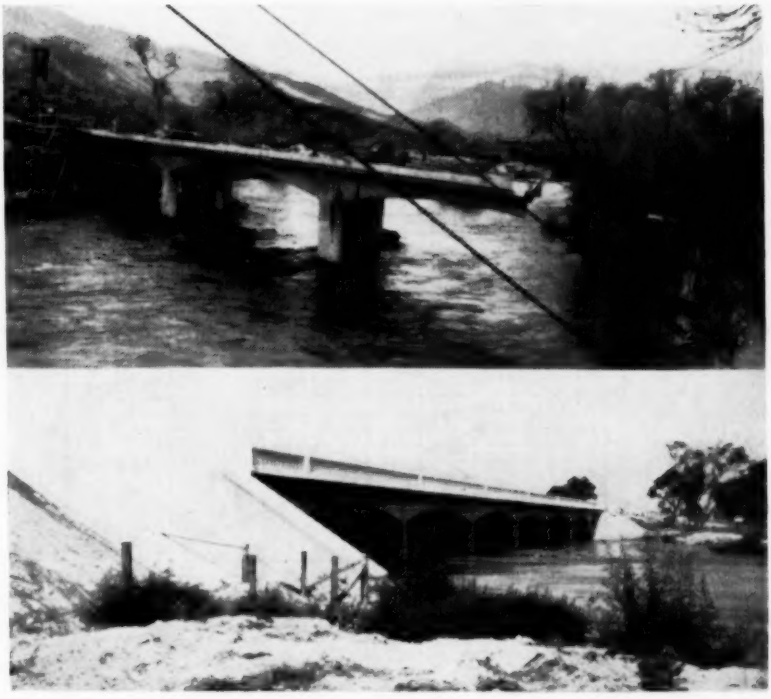


FIG. 14—KERN RIVER BRIDGE AT ISABELLA UNDER CONSTRUCTION

The falsework was washed out during the construction and the picture shows 40 feet of the 60-foot span left hanging in the air. The contractor had stopped his pour at the expansion hinge. No damage was done. The hanging portion was jacked back to grade and the structure completed.

FIG. 15—SANTA ANA RIVER BRIDGE

Six spans of 61 feet with two 20-foot cantilever end-spans. There are two expansion hinges in the structure.

other hinge positions may be made without serious error on a straight line basis, and end moments due to partial uniform loadings easily obtained by finding the area under the appropriate curve.

Occasionally it may be either convenient or necessary to incorporate beams in a bridge for which the haunching is not parabolic nor even symmetrical. In such cases the transformation equations[1] listed in Fig. 24 and 25 may be used to find the desired coefficients for hinged beams using similar coefficients obtained from charts already published for solid beams.[2] To illustrate the use of these transformations

[1]See footnote (1) p. 162.

[2]"Continuous Frames in Reinforced Concrete" by Cross & Morgan, *John Wiley & Sons*, p. 139-155. See also ""Design of Continuous Frames Having Variable Moments of Inertia" by Thor Germundsson, *Civil Engineering*, Oct. 1932, p. 647-8.

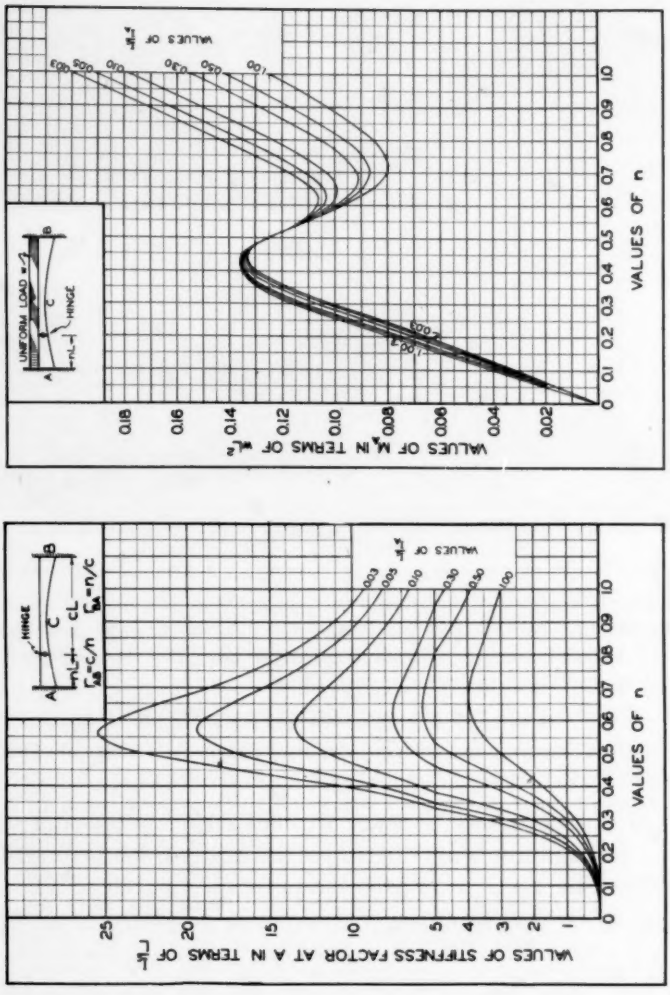

VALUES OF n
VALUES OF M_L IN TERMS OF wL²
VALUES OF STIFFNESS FACTOR AT A IN TERMS OF EI/L
UNIFORM LOAD = w
HINGE
A
B
C
mL
cL
n=c/m
k_B=c/m
k_B=n/c

FIG. 16 AND 17

a beam with a symmetrical parabolic haunch for which $I_C/I_A = 0.3$ will be considered. The following values for the solid beam for stiffness and carry-over factors and for fixed-end moments for a unit load $0.3L$ from the right end will be:

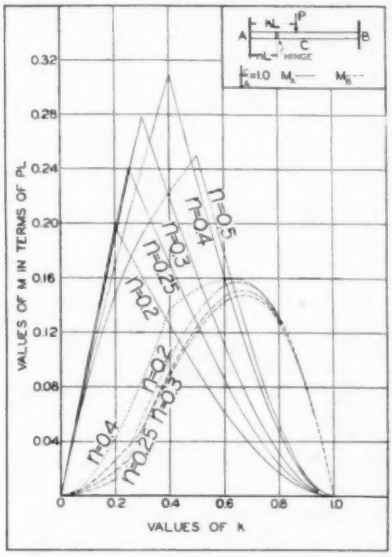


FIG. 18

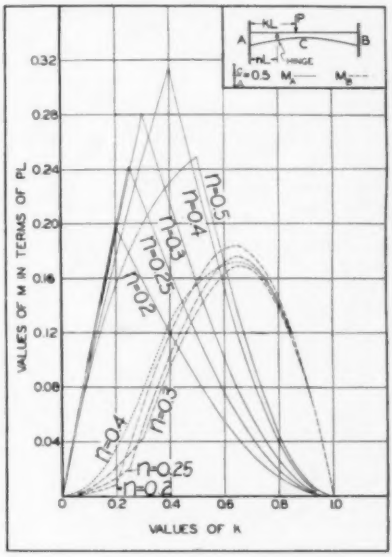


FIG. 19

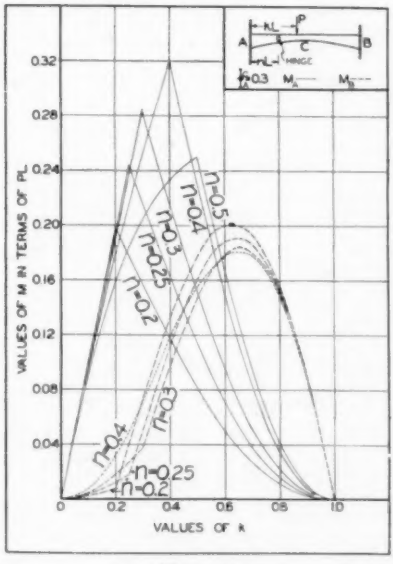


FIG. 20

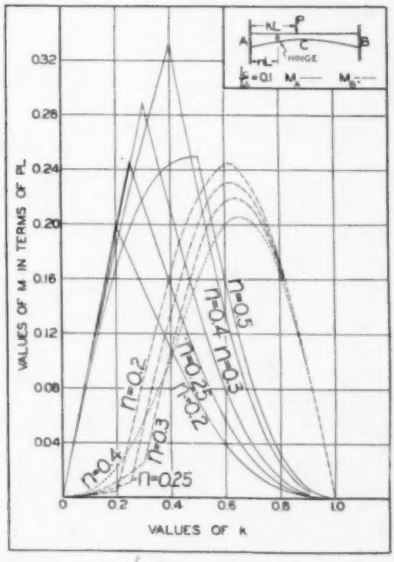


FIG. 21

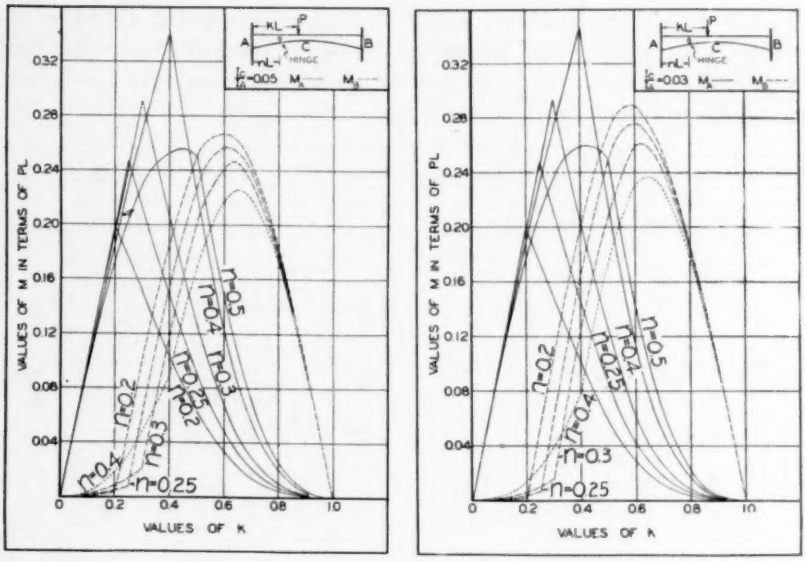


FIG. 22 AND 23

$S' = 7.58$ in terms of I_c/L $\quad M'_A = -0.065$
$r = -0.617$ $\quad M'_B = -0.177$ } in terms of PL

Substituting these values in the equations of Fig. 24 the following results are obtained when the hinge is placed 0.2L from the left end.

$$A = \frac{2}{7.58\,(1 - 0.617)} = 0.690 \qquad i_c = \frac{1}{2 \cdot 7.58\,(1 + 0.617)} = 0.0408$$

$$i_n = 0.0408 + 0.690\,(0.5 - 0.2)^2 = 0.1029$$

$$m_c = 0.0408\,(-0.3 - 0.065 + 0.177) = 0.0408\,(-0.188) = -0.00767$$

$$q = 0.690\left(0.065 - \frac{0.188}{2}\right) = -0.0200$$

$$m_n = -0.00767 - 0.0200(0.5 - 0.2) = -0.01367$$

$$M_A = -\frac{-0.01367(-0.2)}{0.1029} = -0.0266$$

$$M_B = -0.3 - \frac{-0.01367(0.8)}{0.1029} = -0.1934$$

} in terms of PL

$S_A = 0.04/0.1029 = 0.389$
$S_B = 0.64/0.1029 = 6.22$ } in terms of I_c/L

These final values, as calculated, agree very well with those which could have been read directly from the chart of Fig. 20. It should be evident, however, that unless rather exact values can be read from

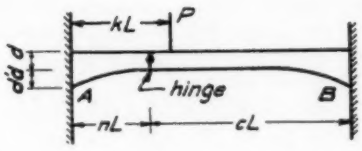
$i_n = i_c + AL^2(0.5 - n)^2$

where $A = \dfrac{2}{S'(1 + r')}$

and $i_c = \dfrac{L^2}{2S'(1 - r')}$

$S'_A = n^2L^2/i_n \qquad S'_B = c^2L^2/i_n$

$r_{AB} = c/n \qquad r_{BA} = n/c$



CONCENTRATED LOAD TO LEFT OF HINGE

$M_{SA} = -PkL \qquad M_{SB} = 0$

$q = A\left(M'_B - \dfrac{-M_{SA} + M'_A - M'_B}{2}\right)$

$m_c = \dfrac{i_c}{L}(-M_{SA} + M'_A - M'_B)$

$m_n = m_c + qL(0.5 - n)$

$M_A = M_{SA} - \dfrac{m_n nL}{i_n}$

$M_B = -\dfrac{m_n cL}{i_n}$

CONCENTRATED LOAD TO RIGHT OF HINGE

$M_{SA} = 0 \qquad M_{SB} = -PL(1 - k)$

$q = A\left(-M'_A + \dfrac{M_{SB} + M'_A - M'_B}{2}\right)$

$m_c = \dfrac{i_c}{L}(M_{SB} + M'_A - M'_B)$

$m_n = m_c + qL(0.5 - n)$

$M_A = \dfrac{-m_n nL}{i_n}$

$M_B = M_{SB} - \dfrac{m_n cL}{i_n}$

Moments are—when bottom fiber is in tension. Carry-over factors are negative

FIG. 24—SYMMETRICALLY HAUNCHED BEAM

$x = \dfrac{-r'_B(1 - r'_A)}{2r'_A r'_B - r'_A - r'_B}$

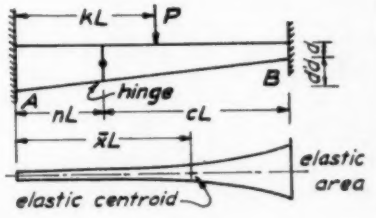
$A = \dfrac{i}{S'_A i - x^2L^2}$

$i = \dfrac{L^2(2x - 1)}{S'_A - S'_B}$

$i_n = i + AL^2(x - n)^2$

$S_A = n^2L^2/i_n \qquad S_B = c^2L^2/i_n$

$r_{AB} = c/n \qquad r_{BA} = n/c$



CONCENTRATED LOAD TO LEFT OF HINGE

$M_{SA} = -PkL \qquad M_{SB} = 0$

$q = A\left\{M'_B - (1 - x) - (M_{SA} + M'_A - M'_B)\right\}$

$m = \dfrac{i}{L}(-M_{SA} + M'_A - M'_B)$

$m_n = m + qL(x - n)$

$M_A = M_{SA} - \dfrac{m_n nL}{i_n}$

$M_B = -\dfrac{m_n cL}{i_n}$

CONCENTRATED LOAD TO RIGHT OF HINGE

$M_{SA} = 0 \qquad M_{SB} = -PL(1 - k)$

$q = A\left\{-M'_A + x(M_{SB} + M'_A - M'_B)\right\}$

$m = \dfrac{i}{L}(M_{SB} + M'_A - M'_B)$

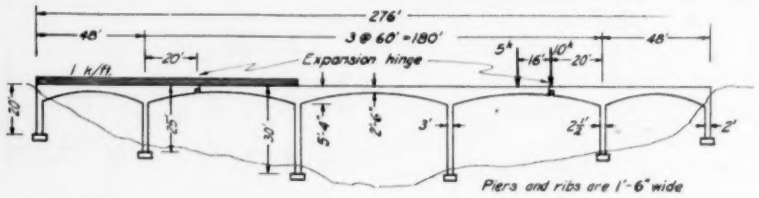
$m_n = m + qL(x - n)$

$M_A = -\dfrac{m_n nL}{i_n}$

$M_B = M_{SB} - \dfrac{m_n cL}{i_n}$

Moments are—when bottom fiber is in tension. Carry-over factors are negative

FIG. 25—UNSYMMETRICALLY HAUNCHED BEAM



Note—Fixed-end moments are plus when they act clockwise on the joint. Sign of carry-over is therefore, also plus.

Values of r	←.709→		2→	←0.5	←.709→		0.5→	←2	←.709→	
	.756	.598	.110	.333	.358	.358	.333	.110	.598	.756
Fix—E. M.	239.0	−239.0	407.0	−358.0	0	0	91.5	−214.5	0	0
	−181.0	−100.5	−18.5	119.3	128.2	−32.8	−30.5	23.6	128.3	0
	−71.2	−128.3	59.8	−37.0	−23.2	90.9	47.2	−15.3	0	90.9
	53.8	41.0	7.6	20.0	21.5	−49.4	−46.0	1.7	9.2	−68.7
	29.1	38.2	10.0	15.2	−35.1	15.3	3.4	−23.0	−48.7	6.5
	−22.0	−28.8	−5.3	6.6	7.1	−6.7	−6.2	7.9	42.9	−4.9
	−20.4	−15.6	3.3	−10.6	−4.8	5.0	15.8	−3.1	−3.5	30.4
	15.4	7.4	1.4	5.1	5.4	−7.5	−6.9	0.7	4.0	−23.0
	5.2	10.9	2.6	2.8	−5.3	3.8	1.4	−3.5	−16.3	2.8
	−3.9	−8.1	−1.5	0.8	0.9	−1.9	−1.7	2.2	11.8	−2.1
	−5.7	−2.8	0.4	−3.0	−1.3	0.6	4.4	−0.9	−1.5	8.4
	4.3	1.4	0.3	1.4	1.5	−1.8	−1.7	0.3	1.4	−6.4
	1.0	3.1	0.7	0.6	−1.3	1.1	0.6	−0.9	−4.5	1.0
	−0.8	−2.3	−0.4	0.2	0.3	−0.6	−0.6	0.6	3.2	−0.8
Beam M	42.8	−423.4	467.4	−236.6	93.9	16.0	70.7	−224.2	126.3	34.1
Col. M	−42.8		−44.0	142.7		−86.7		97.9		−34.1
Col. Shear	−2.47		−1.97	5.22		−3.17		4.38		−1.97
Bent Shear		−4.44			2.05				2.40	

Sidesway Correction—Bent AB

Piers A and F $I/L^2 = 1.385/17.33 = 0.0799$ Column sidesway M = 100.0
Piers B and E $I/L^2 = 2.097/22.33 = 0.0938$ Column sidesway M = 117.4
Piers C and D $I/L^2 = 2.962/27.33 = 0.1083$ Column sidesway M = 135.5

Values of r	←.709→		2→	←0.5	←.709→		0.5→	←2	←.709→	
	.756	.598	.110	.333	.358	.358	.333	.110	.598	.756
Col. M.	100.0	117.4		0		0		0	0	
	−75.6	−70.2	−13.0	0	0	0	0	0	0	0
	−49.8	−53.6	0	−26.0	0	0	0	0	0	0
	37.7	32.1	5.9	8.7	9.3	0	0	0	0	0
	22.8	26.7	4.3	11.8	0	6.6	0	0	0	0
	−17.2	−18.5	−3.4	−3.9	−4.2	−2.4	−2.2	0	0	0
	−13.1	−12.2	−2.0	−6.8	−1.7	−3.0	0	−1.1	0	0
	9.9	8.5	1.6	2.8	3.0	1.1	1.0	0.1	0.66	0
	6.0	7.0	1.4	3.2	0.8	2.1	0.2	0.5	0	0.47
	−4.5	−5.0	−0.9	−1.3	−1.4	−0.8	−0.8	−0.05	−0.30	−0.36
	−3.5	−3.2	−0.7	−1.8	−0.6	−1.0	−0.1	−0.4	−0.26	−0.21
	2.6	2.3	0.4	0.8	0.9	0.4	0.4	0.07	0.49	0.16
Beam M.	−84.7	−86.1	−6.4	−12.5	6.1	3.0	−1.5	−0.9	0.59	0.06
Col. M.	84.7	92.5		6.4		−1.5		0.3		0
Col. Shear	4.89	4.14		0.234		−0.055		0.013		0
Bent Shear		9.03			0.179				0.013	

Similarly, for Bent EF

Col. M.	0	0		0		0		117.4		100.0
Beam M.	0.06	0.59	−0.9	−1.5	3.0	6.1	−12.5	−6.4	−86.1	−84.7
Col. M.	0	0.3		−1.5		6.4		92.5		84.7
Col. Shear	0	0.013		−0.055		0.234		4.14		4.89
Bent Shear		0.013			0.179				9.03	

FIG. 26 (CONTINUED NEXT PAGE)

Bent CD

Col. M.	0	0		135.5		135.5		0	0	
	0	0	0	−45.2	−48.5					
	0	0	−22.6	0	−34.4					
	0	13.5	2.5	11.5	12.5					
	9.6	0	5.7	5.0	8.1	Calculations symmetrical				
	−7.2	−3.4	−0.6	−4.4	−4.7	about center line.				
	−2.4	−5.1	−2.2	−1.3	−3.3					
	1.8	4.4	0.8	1.5	1.7					
	3.1	1.3	0.8	1.6	1.2					
	−2.3	−1.2	−0.2	−0.9	−1.0					
	−0.9	−1.7	−0.5	−0.5	−0.7					
	0.7	1.3	0.2	0.4	0.4					
Beam M.	2.4	9.1	−16.1	−32.3	−68.7	−68.7	−32.3	−16.1	9.1	2.4
Col. M.	−2.4	7.0		101.0		101.0		7.0		−2.4
Col. Shear	−0.139	0.313		3.69		3.69		0.313		−0.139
Bent Shear	0.174				7.380				0.174	

Proportioning sidesway moments

	42.8	−423.4	467.4	−236.6	93.9	16.0	70.7	−224.2	126.3	34.1
AB . *a*	−42.1	−42.8	−3.2	−6.2	3.0	1.5	−0.8	−0.4	0.3	0
CD . *b*	−0.7	−2.6	4.6	9.2	19.5	19.5	9.2	4.6	2.6	−0.7
EF . *c*	0	−0.1	0.2	0.4	−0.8	−1.6	3.3	1.7	22.5	22 .1
Final Beam M.	0	−468.9	469.0	−233.2	115.6	35.4	82.4	−218.3	146.5	55.5
Final Col. M.	0	0		117.6		−117.8		71.8		−55.5
Col. Shear	0	0		4.30		−4.31		3.21		−3.21

FIG. 26 (CONTINUED FROM PAGE 168)

the existing charts for solid beams, the coefficients determined by these equations will be in error.

Values of the fixed-end moments can be found for a series of load points along the beam as indicated above, the moment plotted against the load position as was done in Fig. 18 to 23, and the fixed-end moments due to a partial or total uniform load then determined by obtaining the area under the corresponding part of this influence line.

The use of the charts in analyzing a typical rigid-frame bridge as a whole rather than by parts, when some of the members are hinged, is illustrated below. The bridge used in the example is shown in Fig. 26. The various necessary factors used in analyzing the structure by moment distribution were taken from existing charts[2] for the solid beams, and from the charts accompanying this paper for the two hinged spans. They are listed below.

Beams AB and EF

$$d' = 1.132 \qquad S' = 13.2\frac{2.50^3}{48} = 4.298$$

$$r' = 0.709$$

M' due to uniform load $= 0.1039 \cdot 1 \cdot 48^2 =$ 239 ft.k.

[2]See footnote (9) p. 163.

Beam CD $S' = 13.2\dfrac{2.50^3}{60} = 3.438$

$r' = 0.709$

Beams BC and DE $S_B = 3.05\dfrac{2.50^3}{60} = 0.794 = S_E$

$S_C = 12.3\dfrac{2.50^3}{60} = 3.203 = S_D$

$r_B = r_E = 2$

$r_C = r_D = 0.5$

$M_B = 0.1131 \cdot 1 \cdot 60^2 = 407$ ft.k.

$M_C = 0.0995 \cdot 1 \cdot 60^2 = 358$ ft.k.

$M_D = 0.047 \cdot 10 \cdot 60 + 0.211 \cdot 5 \cdot 60 = 91.5$ ft.k.

$M_E = 0.309 \cdot 10 \cdot 60 + 0.097 \cdot 5 \cdot 60 = 214.5$ ft.k.

Piers A and F S at top $= 3\dfrac{2^3}{17.33} = 1.385$

Piers B and E S at top $= 3\dfrac{2.5^3}{22.33} = 2.097$

Piers C and D S at top $= 3\dfrac{3^3}{27.33} = 2.962$

In the preliminary analysis the fixed-end moments are balanced and carried over in the usual way, but they produce unbalanced shears in each of the three bents. These bents must, therefore, be corrected for sidesway. This is accomplished by deflecting one of the bents sideways with the joints locked, proportioning the moments produced in the columns of that bent according to I/L^2, distributing these moments over the entire structure, and finding the new unbalanced shears. Each of the other two bents is treated in a like manner. The proportional part of these final moments to be added algebraically to those produced by the original loading may be determined by equating the unbalanced shears in each bent, due to the applied load and to each of the induced side-sway effects, to zero. Then,

$$-4.44 + 9.030a + 0.174b + 0.013c = 0 \qquad \text{Bent } AB$$
$$2.05 + 0.179a + 7.380b + 0.179c = 0 \qquad \text{Bent } CD$$
$$2.40 + 0.013a + 0.174b + 9.030c = 0 \qquad \text{Bent } EF$$

Hence $a = 0.497 \qquad b = -0.284 \qquad c = -0.261$

The final moments, together with the solution of the individual parts for sidesway, etc., are shown at the end of the illustrated example. The calculations involved in the moment distribution and carry-over operations of the analysis could have been shortened somewhat by

balancing joints alternately, rather than simultaneously, and writing only the moments carried over.[3]

NOTATION OF SYMBOLS

L = length of beam

d = center (or minimum) depth of beam

d' = haunching factor = $\frac{\text{end depth} - \text{min. depth}}{\text{min. depth}}$

A = elastic area

i_c, i_n, i, = moment of inertia of the elastic area about the beam center, the hinge point, and the elastic centroid respectively.

I_c, I_A = moment of inertia of beam cross-section at center and end A of span.

M_A, M_B = fixed end moments

M_{SA}, M_{SB} = statical end moments—hinged beam made statically determinate and treated as a double cantilever by removal of the hinge.

q = area of the statical moment diagram

m_c, m_n, m, = moment of "q" about the beam center, the hinge, and the elastic centroid respectively.

r = carry-over factor—r_{AB} is the carry-over from end A to B.

S = stiffness factor

P, w = concentrated and uniform loads respectively.

a, b, etc. = constants

Note—Primed symbols refer to solid beams; unprimed symbols to hinged beam.

[3] "Should the Type of Indeterminate Problem Determine Its Method of Solution" by G. A. Maney, JOURNAL, Amer. Concrete Institute, May-June 1938, *Proceedings*, Vol. 34, p. 605.

Discussion, to close in June 1939 JOURNAL, should reach A. C. I. Secretary in triplicate by Apr. 1.

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Discussion of a paper by D. H. Pletta and Leonard C. Hollister

Beams with Intermediate Expansion Hinges in Rigid-Frame Bridges*

BY HOMER M. HADLEY, A. J. BUHLER, A. B. COHEN AND CLOSURE BY THE AUTHORS

ERRATA

In the original paper: The Captions for Fig. 5 and 8 should be interchanged; the bridge illustrated in Fig. 8 is the Cache Creek Bridge and is referred to as Fig. 5 in the text. For Fig. 25, for a concentrated load to the left of the hinge,

$$q = A \left\{ M'_B - (l - x)\,(-M_{SA} + M'_A - M'_B) \right\}$$

and for a concentrated load to the right of the hinge,

$M_{SA} = O$ instead of $M_S A = O$.

BY HOMER M. HADLEY†

If proof additional to the logic of the authors' text were needed to support their advocacy of the single expansion joint in place of the more commonly used suspended span, that proof is certainly to be found in their Fig. 14, the picture of the Kern River Bridge near Isabella after a flood had carried out the construction falsework. A suspended span would inevitably have gone out with the falsework whereas with the single joint the corresponding portion of the deck structure shown at the middle of the picture remained in place.

It is gratifying to observe the widespread change that is coming in expansion joints: the abandonment of what the authors designate in Fig. 1 as Type I and Type II joints and the adoption of the Type III and Type IV joints. In favor of the latter are all the

*JOURNAL, Amer. Concrete Inst. Jan. 1939; *Proceedings* Vol. 35, p. 149.
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advantages enumerated in the paper and also one other which is of some importance: the loads to be transferred through the joint at the contraflexure point are materially less than when the joint is placed at the support.

The authors point out that hinges are difficult and costly to consstruct, are troublesome to maintain and are the source of staining and discoloration if leakage occurs. Their words are: "It is therefore advisable to eliminate unnecessary hinges." This raises the question: when are hinges unnecessary, or conversely stated, when are they necessary?

The spacing of joints may be determined by the explicit or implicit requirements of specifications handed down from On High or by the requirements of Nature. Although it is sometimes suggested that these are identical conditions Nature's mordant sense of humor not infrequently reveals unmistakeable differences between the two. What is of real interest of course is to know the requirements of Nature for as Sir Francis Bacon said: "The only way to command Nature is to obey her."

The authors state that the spacing of the joints throughout the structure depends upon the stiffness of the legs or bents. However the chief variable of which joint spacing is a function is the assumed unit expansion of the concrete per degree change of temperature. If this unit expansion were zero it would not matter how stiff or how flexible the bents were: no joints would be needed. If its true value is one-half or two-thirds that currently assumed then the number of joints could be reduced to one-half or two-thirds the number now employed and undoubtedly bridges would be the better for it.

Without knowing the exact history of the prevailing average coefficient of linear expansion, 0.000,006,5 ± 0.000,000,5, the writer feels safe in assuming that carefully controlled laboratory experiments showed this to be the coefficient of expansion of the specimens of unreinforced concrete tested; that then in no very scientific manner this coefficient was assumed to be the coefficient of expansion of all bridges; that little or no interest has been taken since to see whether this was actually so or not; that what limited data have been published on observed movements in bridges have received little or no consideration from committees charged with the periodical revision of bridge specifications.

Why bridges should have a lesser expansion than laboratory specimens of plain concrete very likely is due to the fact that they are reinforced structures and also because their concrete in drying out

shrinks and develops fine hair cracks at many points along the length of the deck which small shrinkage cracks thereafter provide space in which some expansion finds relief without affecting the overall length of the structure. Whatever the cause the indications certainly are that there is decidedly less expansion in bridges than provision is made for.

Concrete and Constructional Engineering, Jan. 1928, describes the then recently completed George V Bridge at Glasgow, Scotland in considerable detail. This is a continuous box girder structure, stone-faced, with spans 123 ft., 166 ft., 123 ft., and therefore of an overall length of 412 ft. At the abutments and at the two intermediate piers the deck is supported on rockers, so except for the slight frictional resistance of the rockers it is absolutely free to expand and contract. Thus unrestrained, a concluding paragraph states that the maximum observed variation in length, winter to summer, was ¾ in. in the aforesaid 412 ft. of continuous length. Climatically Glasgow is about the same as western Washington or the Southwestern coastal region of British Columbia.

The Jan. 1939 issue of the same magazine describes the general features of the new Waterloo Bridge, now under construction across the Thames at London. This is likewise a stone-faced box-girder structure having five 250-ft. spans with ends cantilevering beyond the abutments. The middle span contains a centrally located suspended section, from the ends of which to the ends of the cantilevers projecting beyond the abutments the deck is continuous upon both sides, i.e., for unbroken lengths of approximately 600 ft. The article describes the box-girder supports, located along the axis of the narrow cellular piers as "bearing walls" and they probably are flexible. Nevertheless the implications of the construction are that expansion and contraction are regarded with calmness and equanimity.

In the *Engineering News-Record* of Jan. 10, 1935, p. 57, La Motte Grover, Office Bridge Engineer, State Highway Commission of Kansas, reports measured expansion and contraction in the two sections of the continuous concrete girder bridge at Marysville, Kansas as being about ⅔ of that which should occur under their assumed coefficient of expansion of 0.000 0067 and the known changes of atmospheric temperature.

In the *Engineering News-Record* of Dec. 1, 1938, p. 693 is an interesting letter by B. J. Garnett, bridge engineer of Spokane, Washington describing their experience with bridges and particularly with the

376-ft. unjointed Freya Street viaduct built in 1928 and showing no ill effects from the omission of the joints.

The Lake Washington pontoon bridge, now under contract, will have reinforced concrete barges or pontoons rigidly connected end to end, the top of the barges being shaped to form the roadway, curbs, sidewalks and railings of the bridge. The joints employ shear keys, heavy bolting around the entire perimeter of the section and a final grouting of all open spaces remaining. A practically continuous structure will result. West of the ship channel opening the length of the joined and connected parts will be 5,471 ft; east of the ship channel 948 ft. Since these sections float in the water there is no friction and the longitudinal anchorage is concentrated at the ship channel end. The magnificient length of the western section makes it a unique measuring rod for determining expansion and contraction and it is to be hoped that measurements will be taken. However the annual range of water temperatures is not great, probably about 25°F.

Nearing completion at Yakima, Washington is the Terrace Heights Bridge, a box-girder structure across the Yakima River. It consists of 4 spans of 130 ft. and end cantilevers of 40 ft. The overall length is therefore 600 ft. Its deck is integral and monolithic with the center pier, is supported on concrete-filled steel pipe rollers on the other piers. From end to end there are no joints whatsoever in the deck itself. At the ends small baffle walls retain the approach fills, keeping them out of contact with the bridge structure, which therefore will freely respond to changes of temperature, the only restraint being the frictional resistance of the rollers. It is planned to take careful measurements of the temperature movements in this bridge over a term of years and since the thermal range is from 10° below zero in winter to 100° above in summer, some interesting data should be obtained which will likewise be valuable if any use is made of it. This bridge has a 24-ft. roadway, two 3 ft. 6 in.-sidewalks, steel railings, 5 piers bearing directly on compacted gravel at a depth of 10 to 12 ft. below water surface, untreated concrete surfaces poured against plywood, spans as above noted and was placed under contract at a price of $4.03 per sq. ft., based on its 32-ft. overall breadth. As stated it is of box-girder construction.

Yet another way of dealing with thermal changes, certainly applicable to shorter lengths of structure, say up to 150 or 200 ft., is to build the bridge continuous, provide some form of inexpensive sliding bearings at the ends such as plates separated by asbestos packing and let the deck bear squarely against the end enbankments. This con-

struction may appear somewhat heterodox and unconventional but will work in a highly satisfactory manner. Concrete as designed today is overflowing with unused longitudinal compressive strength which may well be employed to some good purpose. If this proposal sounds preposterously extravagant to any one let him consider how it used to be absolutely necessary to have joints in railroad rails every 33 ft. and that now the absolute necessity of yesteryear has gone *pouf:* the rails are being welded together in mile lengths.

BY A. J. BUHLER*

I should like to point out some further criticism of the four types of structures shown by Fig. 1.

The type I has the only disadvantage that the design results in irregular reinforcing on account of the equalized moment and shear

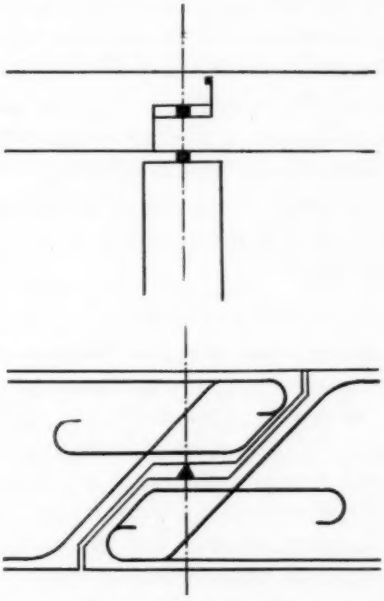

FIG. 27 (TOP) AND 28 (BOTTOM)

surfaces. The increase in size of the center pier can be avoided if the expansion joints are superinposed as in Fig. 27, but this arrangement complicates the design.

Type II may have an advantage if the split-pier can be utilized as esthetical means for subdividing long structures.

*Berne, Switzerland.

Type III has a great advantage on account of maintenance work. It will be possible to raise the suspended span and inspect or, if necessary, repair the hinges.

Type IV effects the elimination of one hinge. The reasons mentioned in favor of the reduction are more or less unportant. But very often long continuous structures are not desirable, because an intermediate hinge gives no complete subdivision, or in other words, does not interrupt continuity. Any sort of defect influences not only a limited part of the structure, but the whole structure. Complete subdivision may also be important from the standpoint of erection (shrinkage, temperature).

If one is convinced that a rigid-frame bridge cannot be built without any hinge he will be obliged to find a good solution for this detail, which should be watertight and easy to maintain. This task is a very hard one, because if the designer has found the solution, he must be sure, that also the execution will be irreproachable. If the number of hinges is not increased type III and IV are equally good.

I believe a good solution may be found if the hinges are built with oblique sections as indicated in Fig. 28 permitting proper placing of the principal reinforcing bars without difficulty.

We adopted such a detail in the design of the rigid-frame bridge of a new 4-track viaduct near Berne (18 spans of 90 ft. each) having only one girder for each track.

BY A. B. COHEN*

I have not had sufficient time to review the theoretical considerations of this paper. This discussion refers particularly to the practical considerations and the economic value derived in maintaining continuity through these hinges. If the practical considerations in the development of the structural details of the hinge are inconsistent with or not equal in value to the structural medium of the frame or body of the bridge, then any economy claimed for the composition will not be realized. These practical considerations have been given careful thought and the speaker will limit his remarks to them at this time.

Reinforced concrete has outstanding advantages compared with structural steel in bridge design where the peculiarities of the site do not prohibit the use of concrete. The advantages not only concern the factors of durability, low maintenance costs and susceptibility in design to provide for a greater range or increase of live load at very little additional cost, but they include the initial cost. A lower initial

*Consulting Engineer, N. Y. C. Mr. Cohen's discussion, presented at the 35th Annual Convention is published as supplemented in writing.

cost is easily realized in concrete bridges for four-lane roadway capacity and by careful design can also be realized in structures of less width or travel capacity.

The advantages gained by use of reinforced concrete in the matter of durability and saving in maintenance appears to be lost in an incongruous combination of structural steel and reinforced concrete of the hinge design under consideration. From the standpoint of impairment, the hinge is more vulnerable in this concrete design than it would be if placed in a bridge frame of structural steel. In the latter it could readily be replaced in case of failure or deterioration. Embedded in the concrete frame, it cannot be replaced. The requirement of replacability is based upon the premise that it is practically impossible to detail a water tight joint subject to both vertical and horizontal movements. The rubber latex asphalt filler used to caulk the joint, as indicated in Fig. 2 of the paper, will not prevent seepage of water through the joint even though it is maintained at regular intervals. Therefore the structural parts of the hinge are subject to rusting and deterioration, the acceleration of which depends upon the severity of climatic conditions.

The concrete in which the hinge is embedded is subject to an action that might destroy the hinge in less time than the rusting action of the severest climate. This action is the vulnerability of the concrete adjacent to the hinge to cracking and disintegration due to the pounding it receives by passage of heavy trucks. This action is accelerated when a slight unevenness develops at the joint due to construction faults or subsequent uneven settlement of adjacent parts.

A recent development of an expansion joint detail of a bridge designed in the speaker's office is described in an article pertaining to the Ferry Street Bridge built at Binghamton, New York, which appeared in the *Engineering News-Record* of April 28, 1938. The concrete surrounding this joint is armored by a cast iron grill. All concrete surfaces of the joint subject to drip of drainage and therefore to surface disintegration has a protective covering of sheet copper. All parts of the structural steel composing the joint are replacable.

There is no indication in the cross sections of the California bridges that an allowance or provision has been made for surface disintegration or wear of the bridge slab itself—no added thickness to the slab, or no pavement surfacing that can be replaced. Thix creates an additional factor of weakness at the joint in its ability to resist the gradual crumbling of the concrete due to pounding of truck wheels. When this happens, the joint will be irreparable. On all important struc-

tures, it appears to be advisable to cover the slab with a waterproofing membrane and a sheet paving of either concrete, or a bituminous compound so that the construction joints are protected from the ravages of water and the pavement can be replaced or repaired. The speaker's office has also discarded the seemingly apparent economy of a cantilever sidewalk, as shown in Fig. 3 of the paper, which is considered unsafe in these days of fast and heavy traffic equipment, and which precludes a substantial termination of the waterproofing membrane. This is another theoretical economy that must be sacrificed for practical considerations of exceptional value.

Referring to the four types of expansion joints for continuous span structures, as shown in Fig. 1 of the paper, the speaker's experience has long ago led to the abandonment of the sliding joint—type I. It was found that there was always the possibility of the joint "freezing up," so to speak, by an unevenness due to faulty construction or by rusting of the sliding parts until the frictional resistance of the sliding parts was greater than the strength of the beam or slab in shear. This resulted in the cracking of the beam or slab adjacent to the support. Type II, however, has been used very effectively from a structural and architectural point of view, in which the movable parts of an expansion joint have been eliminated. This lends itself also very effectively to architectural treatment. By very little additional expense, the joint can be hidden entirely from view by a pilaster built integrally with one of the columns and free from contact with the other. The expansion hinge of types II and III architecturally and otherwise, interrupt the continuity more than the double pier because these joints must be carried through the railing at an unsymmetrical point, which even to the casual eye makes the joint more noticeable architecturally and more unsightly than the double column arrangement.

On certain types of expansion joints, we found it expedient and necessary to incorporate troughs and down-spouts underneath the expansion joint in order to prevent disintegration of the lower concrete surfaces by water dripping through the joint. The troughs, hidden from view, were formed in a concrete coping, and the down-spouts were carried through the fill behind the abutments to a French drain in the ground in order to prevent freezing of the water dripping down through the pipe, which often occurs when the end of a down-spout is exposed to the air.

In closing the speaker calls attention to the low cost per square foot of the California bridges as recorded in this paper. In view of the speaker's opinion that the bridge decks in question have not been

developed to the full potentialities of a reinforced concrete bridge, which should be a symbolism of permanence, these costs must be proportionately appraised.

AUTHORS' CLOSURE

The discussion contributed by Mr. Hadley dwells mainly on the difference between actual and calculated expansion in rigid frame bridges and questions the advisability of incorporating any expansion joints. There is not a great deal of quantitative data on the phenomenon available at present, perhaps too little to neglect the effect entirely in design. For this reason the data supplied by Mr. Hadley is most welcome. The whole subject needs intensive field study.

Probably, a great deal of the past grief resulting from temperature effects has been due to improper design, or, in other words, to poorly placed reinforcing. Many structures have been designed or reinforced for "free-end" conditions for the various members, when in effect they were constructed as "fixed-end" members. For example, it is now being found advantageous to spread the negative reinforcing steel out into the roadway slab instead of bunching it at the girders as was formerly the custom.

There seems to be some concern expressed in regard to the maintenance of this type of hinge. It is suggested that it would be very difficult to replace the rollers of the expansion joints on this type of girder. It is possible that other localities have more need for replacing expansion joint details than is called for in the regions in which those illustrated in the paper were used. If so, and if any designer felt the need for such a provision, it could be easily provided for—that is, provisions could be made for the possible removal of the rockers. However, the remainder of the expansion joint details which are anchored to the concrete would be very difficult to remove in any type of design.

The detail cited by Mr. Cohen minimizes the exposure of such details, but it is incorporated in the floor of a spandrel arch bridge and not in the main bridge member. It is apparently designed to transmit no shear, thrust or moment and would, therefore, be slightly more susceptible to traffic impact than the expansion hinges illustrated in the paper which do transmit shear only.

Mr. Buhler's discussion has also brought out some interesting points. His suggestion of an oblique hinge has merit and deserves some thought and study. He has also pointed out the fact that provision for expansion as shown in Type III, has the advantage of making it possible to construct a bridge in separate and independent units—his

thought being, apparently, that in case of damage to one portion of the structure, it would not necessarily carry through and affect the entire structure. There is some merit in the thought, but this fear, which has been embedded in the minds of engineers as regards unequal settlement in the case of continuous structures, is somewhat imaginary and overrated. Two continuous structures, each of which had one of their bents undermined by the scour of an excessive flood will serve to illustrate this point. In one case a reinforced concrete girder structure, consisting of three spans at 50-feet and two 18-foot cantilever end spans had one of the end piers undermined, causing a settlement of 9 inches. No apparent damage was done to the girders, and they were cut loose from the settled bent and jacked back to grade. Today, except for the new concrete which was necessary to raise the pier, it is impossible to notice anything has ever happened. In another case, consisting of a continuous girder structure composed of two central spans at 120-ft. and two end spans at 100-ft., the center pier was undermined due to an unprecedented flood and settled 1 ft.-4⅝ in. on the upstream end and 1 ft.-5¾ in. on the downstream end. The structure was jacked back to its grade without interruption to traffic, and in both cases, except for the continuity and unity of the structures as a whole, the piers would have been completely lost and also the spans supported by them.

During the unprecedented floods of March, 1938, very little damage was done to the new modern bridges of the continuous type, while many of the older designs of simple span construction suffered considerable damage due to undermining of piers resulting in final collapse.

Review of the performance during severe floods of continuous structures built integral with piers seems to indicate that such unity presents a considerable factor of safety against complete collapse. It is possible in such severe cases as the Vicksburg-Jackson highway bridge mishap that a continuous monolithic type of structure would have prevented complete collapse to such an extent that the dozen automobiles would not have plunged into the river.

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Mixer Efficiency or Mortar-Mix Tests*

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SYNOPSIS

In this paper, to maintain production schedules on Coulee Dam meant two things: changes in and elimination of mixer blades and a mortar-mix test schedule whose results, shown by an "efficiency factor" indicated necessity for more or less mixing time. This paper tells how these problems were met.

The Coulee Dam mixing plants are each equipped with four 4-yard mixers of a new type. The specifications provided for not less than 2½ minutes mixing. When the plants started the batches did not appear to be well mixed at 2½ minutes, and longer mixing, up to 5 or 10 minutes or more at times was required. To satisfy the contractor that this longer mixing was necessary, samples of mortar were taken from several places in the mixer after 2½ minutes mixing and were analyzed for comparative sand, cement, and water contents. These tests showed at times as much as 250 per cent variation in s/c and w/c ratio in different parts of the mixer although the appearance to the operator was not materially different. Similar results have since been obtained in other standard type mixers.

A more rapid method of making mortar mix tests was then developed in which mortar samples from different parts of a batch were unscrambled by washing and immersion, in a manner (described later) somewhat similar to the Dunagan method of mix analysis.[1] The contractor was then required to mix beyond the specified 2½ minutes only sufficient time to produce the required uniformity. This required about 5 minutes at first, but finally after considerable adjustment of batching sequence and refinements of control as well as some

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[1]JOURNAL Amer. Concrete Inst., Dec. 1929, *Proceedings*, Vol. 26, p. 202.

blade changes which involved discarding one-third of the blades, they were allowed to cut the time to the specification minimum of 2½ minutes.

In the late summer of 1936 it became evident that it would be necessary to mix concrete more rapidly than was possible with 2½ minute mixing period specified, if the river diversion were to be made that season. The contractors, therefore, asked for a decrease of 15 seconds in the mixing period to enable them to meet this emergency, and it was possible to grant this concession subject to their maintaining the mixing efficiency within the required limits of 10 per cent variation as indicated by regular mortar-mix tests. This they were usually able to do and the season's work was finished in time for the desired river diversion which saved them a season's delay. During the winter shutdown they built a 1/3 scale model of the plant mixers and sent it to the laboratory to use in experimenting with blade changes. Using the mortar-mix test method to check results, many blade variations were tried out, and as a result another 1/3 of the blades were discarded. With the three remaining blades of somewhat altered shape much better results were secured than before, as the mixers now mixed more rapidly and also discharged much more quickly.

In 1937, due to a bad cofferdam leak, and unexpected foundation conditions, the contractors again found their schedule lagging so that it was doubtful whether the concrete could be brought up to the required height that season. At their request, and under the same restrictions as to mortar-mix test fluctuations, they were again allowed to cut 15 seconds more from the mixing time, and some more world records were broken in completing the 1937 work.

Periodically, the mixer performance is checked by a computed efficiency factor (more fully described later) which is the average of the percentage of all the tests in which the mix variation comes within 10, 15, and 20 per cent. These factors are kept for each mixer and averaged for each plant. The first month the mixing plant efficiency at 2½ minutes mixing time was 47 per cent, after batching sequence improvements it rose to 75 per cent, and after the first blade improvements it advanced to over 90 per cent. Since final blade changes in 1937 it reached 98 per cent or better with only a 2-minute mixing period.

It is significant that these improvements in mixing show up in the cylinder test uniformity factor. This factor improved from 88 per cent in 1935 to about 96 per cent in 1937.

To secure consistent results requires close control of sampling and cylinder tests and regular and frequent mortar-mix tests, as it has been found that very slight variations in batching sequence or other factors may vitally affect mixing efficiency. The test method can undoubtedly still be shortened and improved and furnishes a convenient tool which should help solve many other interesting and important problems in concrete manufacture.

THE MORTAR-MIX TEST

The mortar-mix test now in use was based on the assumption that if the sand, cement and water were uniformly mixed throughout a concrete batch, and there was no noticeable segregation of coarse aggregate, the mixing was satisfactory.

A standard procedure at Coulee Dam is to run a mortar-mix test from each active mixer once each 8-hour shift. Should any test show a s/c or w/c variation of over 10 per cent between the three samples taken from a mixer a second test of that mixer is run. If the second test is also beyond the 10 per cent limit the mixing time is increased for that mixer, or for all mixers if the trouble is thought to be in the batching system rather than in the individual mixer. The increase made in mixing time depends on the amount of variation shown, and runs from 15 seconds extra for 11 to 15 per cent variation to 1 minute or greater increase for a 20 per cent or over variation.

Test Procedure

Originally the mixers were stopped at the end of the prescribed mixing period, and 3 small samples were selected from front, centre and back of the mixer respectively.

The present procedure is to secure quickly 3 successive samples during the short period of mixer discharge. Samples taken yield about 1000 gr. of mortar.

The samples are at once wet screened through a No. 4 screen into separate pans. Both screen and 3-sampling pails have been previously "primed" by a "dummy" batch taken just prior to the test batch. This was found necessary because any clean utensile will rob cement from the mortar and materially increase the s/c and w/c ratios.

After wet screening, the air weight of each sample is obtained by weighing it in the pan. Each sample is then transferred to a small pail previously counter-balanced in water, and the pail and sample is carefully immersed and the immersed weight of each sample is obtained.

Samples are then thoroughly washed on a one hundred mesh sieve and the cement and minus one hundred sand are removed. The remaining sand is weighed again in water, and from the original weight in air and the two immersed weights the original air weights of dry sand and cement, and the original weight of water in the sample are computed.

An approximate correction in the apparent cement loss is made by using a correction factor of 8 per cent or so, for minus 100 mesh sand which is washed out with the cement.

U. S. BUREAU OF RECLAMATION—GRAND COULEE DAM

CONCRETE MIX ANALYSIS

Plant: East Mixer: South Date: Sept. 30, 1937 Shift: Nite
Inspector: Craig Mortar Tester: Coogan
Concrete Mixing Time: 2 min. Mortar Test Taken After: 2 Minute Mixing
Hour: 3:15 Batch No.: S-85061 Slump: 2¼ in. Consistency: 29
Mix No.: $\frac{1\text{-}2.5\text{-}7.2}{6AGRR}$ Max. Size Aggregate: 6" Plant W/C: .564

Scale Weights	Location in Mixer Front	Center	Back
(a) Air weight of pan and mortar ("A" counterweight)	1250.8	1372.8	1409.5
(b) Immersed weight of mortar sample ("B" counterweight)	709.7	775.1	796.9
(c) Immersed weight of sand (No. 4 to +100)	453.1	496.5	515.4
Elapsed time	40 min.	35 min.	30 min.
COMPUTATIONS			
*(d) Immersed weight of sand corr. for 8% (−100) $=\frac{c}{0.92}$	492.3	539.0	560.0
(e) Immersed weight of cement corr. for sand and silt −100 $=b-d$	217.4	236.1	236.9
(f) Air weight of sand (d x 1.581)	778.0	852.0	885.0
(g) Air weight of cement (e x 1.49)	324.0	352.0	353.0
(h) Air weight of water ($a-(f+g)$)	148.8	168.8	171.5
MIX SUMMARY			
Sand/Cement $= f/g$	2.40	2.42	2.51
% of lowest S/C	100	101	105
Water/Cement $= h/g$	.459	.479	.486
% of lowest W/C	100	104	106

Remarks:

*Use 5% if percent of −100 sand is not known.

FIG. 1

Conversions to dry air weight are then made by multiplying the immersed weights of the sand and cement by the factors obtained by dividing the specific gravity of each by its specific gravity minus one. Thus the cement factor used is $\frac{3.04}{3.04-1} = 1.49$ and the sand factor is $\frac{2.72}{2.72-1} = 1.581$.

The lowest s/c ratio is taken as 100 per cent and the others computed from this. Likewise with the w/c ratios.

A typical mortar test illustrating the form used is shown in Fig. 1. It is noteworthy that extreme accuracy in the factors used or in silt content of sand is not essential, as such variations should affect all three samples alike and if the mixing is perfectly done all samples should give the same results.

THE MIXING EFFICIENCY FACTOR

For periodic checks of the mixing efficiency, as well as to watch its reflection in the uniformity of test cylinder strengths, efficiency factors for each mixer and each plant, as well as a control factor for the uniformity of the compression tests at 8 hr. and 28 days, are computed by the formula $\frac{a+b+c}{3} = E$ in which for mortar tests: (a) is the percentage of all tests for any period (as 1-month) which do not show a variation of over 10 per cent; (b) the percentage of all tests

GRAND COULEE DAM—MONTHLY SUMMARY—MIXER TESTS SEPTEMBER 1937

	N		E		S		W	
	S/C	W/C	S/C	W/C	S/C	W/C	S/C	W/C
Average of maximum variation in mixer %	4.5	3.1	4.9	3.5	4.4	3.1	3.8	3.2
Maximum individual variation in mixer %	10	8	14	15	15	15	12	8
% within 10% variation (a)	100	100	90.1	95.8	98.5	98.5	97.0	100
% within 15% variation (b)	100	100	100	100	100	100	100	100
% within 20% variation (c)	100	100	100	100	100	100	100	100
Efficiency Factor of each mixer $\frac{a+b+c}{3}$	100	100	96.7	98.6	99.5	99.5	99.0	100

Average Efficiency Factor..... 99.2 S/C 98.8 W/C 99.5

CONTROL—CYLINDER BREAKS—MASS CONCRETE
6" Max. Wet Screened to 1½" Max.

	8-hour	28-day
Number of Cylinders	67	134
Average Strength	1605	5395
Maximum Strength	1965	6400
Minimum Strength	1220	3750
% within 10% of Average (a)	62.7	75.4
% within 15% of Average (b)	85.1	93.3
% within 20% of Average (c)	95.5	97.8
Control Factor $\frac{a+b+c}{3}$	81.1	88.8

FIG. 2

which do not show variation over 15 per cent; (c) the percentage not over 20 per cent.

For compression test uniformity, the control factor is based on similar percentages of all individual compression tests during the period, (with any specified mix) which do not vary from the average strength for the period more than the above allowance (a, b and c). A typical monthly mixing plant report is shown in Fig. 2.

An interesting and significant side-light developed from these tests was that the s/c and w/c ratios as batched and theoretically present in the mixer, do not appear in the test results. This is mentioned in order to relieve any one of feeling concerned over such results. There are several reasons for this, which are not directly pertinent to this discussion, but which have led to some further interesting research. One of these reasons is that there is a considerable increase in fines during mixing due to grinding action, which is variable. Another reason is that the coarse aggregate consistently

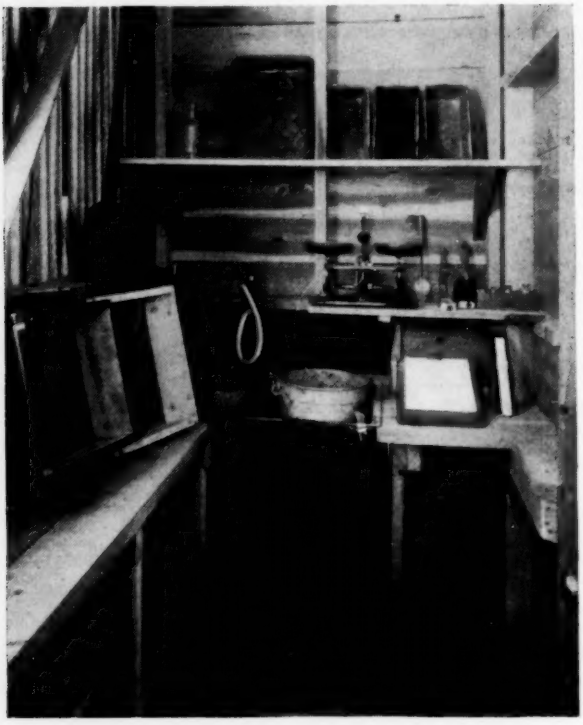

FIG. 3

robs cement from the mortar. The coarse aggregate when screened out from the test sample is coated with a much richer mortar, and at a lower water ratio than the mortar left and used for the test.

A mortar test of 3 samples can be run in about 30 minutes, so in starting a new plant a batch may be sampled and the mixer stopped until test results are obtained, or if not feasible, mixing can be continued on a safe basis until a test is available and then the time shortened as indicated by the test results. Fig. 3 shows the mortar test laboratory at the Coulee Dam Mixing plant.

Discussion, to close in June 1939 JOURNAL, *should reach A. C. I. Secretary in triplicate by Apr. 1.*

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Discussion of a paper by O. G. Patch

Mixer Efficiency or Mortar-Mix Tests*

BY DUFF A. ABRAMS, CLOSURE BY THE AUTHOR AND BY ARTHUR RUETTGERS

BY DUFF A. ABRAMS†

Mixing is one of a dozen steps in the manufacture and placing of concrete. The batching and mixing plants at Grand Coulee Dam are remarkable pieces of equipment; it is unfortunate that the excellent performance of these machines was misunderstood and penalized by unscientific tests. This test method has been promulgated for general use by the Bureau of Reclamation in its "Concrete Manual."

MIXING SPECIFICATIONS

The following is from the 1934 specifications for Grand Coulee Dam which was in force at the time of the tests reported:

Capacity of Mixer	Time of Mixing
2 yards and less	1½ minutes
3 "	2 "
4 "	2½ "
5 "	2¾ "
6 "	3 "

There is no doubt of the right of the Bureau to demand that a 4-yd. batch be mixed 2½ minutes; however it should be stated that there is no rational basis for such a requirement or for increasing mixing time with increase in mixer capacity.

Mix AG is that used by Patch; 43 per cent of the total aggregates consists of 1½-6-in. gravel, the highest of any of the numerous combinations. Mix *A* is about the average.

*JOURNAL Amer. Concrete Inst. Jan. 1939; *Proceedings* Vol. 35, p. 173.
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CONCRETE BATCHES

Below are representative batches used at Grand Coulee:

Material	4-Yd. Batch—lb.	
	AG	A
Cement	1505	1505
Water	850	—
Sand	3760	3760
Gravel 4–¾	2410	2700
Gravel ¾–1½	2170	2140
Gravel 1½-3	2060	2850
Gravel 3-6	4200	3150
Total	16955	

METHOD OF SAMPLING FOR MIXER TESTS

The method of sampling was given scant attention in the paper. The author states: "Originally the mixers were stopped.....The present procedure is to secure quickly 3 successive samples during the short period of mixer discharge."

This is a vital factor, since mixer efficiency hinged on one small sample. Grab samples taken from a mixer are notoriously erratic.

The 1938 "Manual," under "Methods of Sampling and Testing" (p. 291) states:

> It is difficult to obtain representative samples of concrete as it is discharged from the mixer and this shall not be attempted unless arrangements have been made to catch the entire flow for a short period. It is possible to obtain fairly representative samples from certain types of mixers after the drum has been stopped, by the use of a long-handled shovel. When this method is used, small quantities of concrete shall be taken from several points and mixed.

The same method appeared in the 1936 Manual. Was the standard method followed at Grand Coulee?

Improved mixing seems to have been coincident with a change in sampling; possibly the improvement resulting from taking out two-thirds of the blades was related to this change.

TEST PROCEDURE

The basis of the tests and computations is given as follows:

> The mortar-mix test now in use was based on the assumption that if the sand, cement and water were uniformly mixed throughout a concrete batch, and there was no noticeable segregation of coarse aggregate, the mixing was satisfactory.
>
> An approximate correction in the apparent cement loss is made by using a correction factor of 8 per cent or so, for minus 100 mesh sand which is washed out with the cement.

The three small samples of mortar taken during the discharge of the mixer were for an unknown reason designated Front, Center and Back. Each sample was caught in a previously "primed" pan, and then wet-screened through a primed No. 4 sieve.

The following values were determined for each sample:

a. Weight in air.
b. Weight immersed.
c. Weight immersed after washing fine material through a No. 100 sieve.

A mix summary was computed in terms of s/c and w/c which required a correction for the quantity of sand 0-100.

The writer has been unable to find one redeeming feature in the methods of computing or interpretating the results from the original data.

BASIS OF COMPUTATIONS WRONG

Computations based on sand 0-100 ranging from 20 per cent to minus 10 per cent gave values for "per cent of lowest" s/c and w/c which agree with those given for 8 per cent sand. The writer became intrigued by a method of such versatility and assumed specific gravity of sand ranging from 2 to 10,000 and found values which also agree with those in the paper.

The author states that "The coarse aggregate when screened out from the test sample is coated with a much richer mortar, and at a lower water-ratio than the mortar left and used for the test." The average computed w/c was .474, whereas the original batch had .564. How can wet-screening leave both parts of the batch with a lower w/c than the original?

It is apparent that the computed values for sand, cement and water are not the correct ones; they are based on 8 per cent 0-100 sand, whereas the true value may have been 16 per cent, due to the grinding in the mixer. Computed s/c and w/c are known to be wrong.

The author states that his test is ". . . somewhat similar to the Dunagan method of mix analysis." The sole purpose of Dunagan was to determine the actual weights of the constituent materials in a batch; to use values which are known to be grossly in error constitutes a travesty of the method.

COMPUTATIONS BASED ON A FALLACY

The assumption of 8 per cent 0-100 sand as a means of computing the amount of cement seems quite harmless; however this in reality consists of three different assumptions and conceals a fallacy which throws serious doubt on all the conclusions from the tests.

How can one fine material in a batch be uniformly mixed without the other fine materials being uniform also?

The case may be stated:

(1) We assume the concrete is not uniformly mixed and take 3 samples to learn how far it deviates from uniformity. (The paper shows "per cent lowest s/c" of 105, whereas perfect mixing would have given 100 per cent).

(2) In order to compute s/c, a correction of 8 per cent was made in the 0-100 sand in all 3 samples; this assumes:

(a) That the value chosen was immaterial.

(b) That it was exactly the same for the three samples.

We saw that (2a) was harmless; the fallacy lies in (2b).

When we assume the sand uniformly mixed to the extent that there is no variation in the 0-100 size, we have assumed an impossible condition.

A 4-yd. batch contains 3760 lb. sand. In spite of any amount of mixing of a given lot of sand, there is no certainty that 3 grab samples would show a spread of less than 1 per cent through the 100 sieve.

The paper is quite confusing as to what was meant by 8 per cent sand 0-100. From the round number used, we would be compelled to conclude that 8 per cent includes the range 7½ to 8½ per cent; however the author has considered the value correct to 4 significant figures; otherwise it would be impossible to carry the computations to 4 places.

What would be the effect on computed mixer efficiencies if we considered that 8 percent* means anything from 7½ to 8½ per cent? Suppose we assume that all three samples are exactly the same, except that the sand 0-100 varies within the limits 7½ and 8½ per cent. We then compute, according to the paper:

Sand 0-100	7½	8	8½
Per cent of lowest s/c	100	102	104

It should be made clear that we are discussing only the computed spread in mixer efficiency which may be due to errors in sampling; we did not change any of the weighed quantities.

The computed spread in s/c due to this insignificant range in sand 0-100 is nearly equal to the average for September (4.4 per cent).

It must be remembered that this spread of 4 per cent may occur in any sample. The lowest s/c, which the author terms 100 per cent may be 96 to 104 and the highest, 105, may be 101 to 109; so that in this test the computed s/c may be 100 to 114, depending solely on variations in 0-100 sand within the limits of the assumed value of 8 per cent and within the limits of the sampling method.

The "Concrete Manual" (p. 344) used 7½ sand 0-100 with the following note: "The 7½ correction is determined from silt tests on the sand contained in various batches taken during the shift. When the percentage of sand passing a No. 100 standard sieve is not known, use 5 per cent." It is notable that "silt" here includes sand 0-100, whereas every other usage of the term silt in the Manual links it to deleterious substances or contaminating materials.

When we assume the sand uniform for all three samples, we have assumed either (1) that the batch was already uniform, or (2) that the mix was uniform only within the range of our assumption which may cover a spread of 14 per cent. This spread of 14 per cent has no significance whatever so far as uniformity of mixing is concerned; it may explain the 14 and 15 per cent which frequently appear under the "Maximum individual variation in mixer" in Fig. 2.

The author has penalized the mixers for errors concealed in sampling and which could probably not be eliminated by any practicable amount of mixing. When 8 per cent 0-100 sand was assumed for all samples (correct to 4 places) this assumed also that the batch was already uniformly mixed.

METHOD OF COMPUTATION UNSOUND

There is the appearance of accuracy and finality in the computations which do not exist. The use of specific gravity factors to 4 significant figures should be contrasted with the fact that almost any imaginable specific gravity might have been used.

We pointed out above that no value after the introduction of .92 is accurate to more than two significant figures. It can be accepted as axiomatic that no operation in the manufacture, placing or testing of concrete justifies more than three figures; generally two are enough.

COMPUTATIONS UNNECESSARY

It will be shown below that an ideal measure of mixing can be secured from the test data, without complex calculations.

NOMENCLATURE MISLEADING

We find "Air weight of cement," w/c, etc., opposite items which are known to be incorrect; almost any values could have been secured by assuming other percentages of 0-100 sand; yet the actual values could not possibly be altered by such an assumption.

RESULTS IN UNFAMILIAR TERMS

There are well-recognized methods of indicating the variation of a set of observations; such as mean variation, probable error or standard deviation. The author discarded all these for a method which has the following novel features:

(a) No balancing of positive against negative errors.

(b) The *middle* value (nearest to the proper one), is given no weight, except to establish a sequence.

(c) No use is made of the *average* value.

(d) What is termed "variation" should be "spread."

(e) The spread appears to be three to ten times the mean variation.

(f) Mixer efficiency hinges on *one test* (not three); a difference in either high or low value determines the computed result.

(g) In combining the values as in Fig. 2, most of the tests are counted three times, giving a false measure of variation.

TESTS MISINTERPRETED

There is considerable ground for the view that all errors attributed to the mixers at Grand Coulee were due to slight variations in the amount of 0-100 sand, within the limits of the accuracy of sampling. The evidence available indicates that the concrete was probably grossly over-mixed when the samples were taken after two minutes mixing.

There is nothing in the report to show that the concrete was not sufficiently mixed at 1 min.

It is not clear why a mixer which performed satisfactorily at 4 a. m. should not do so at 4 p. m.; or why one which was satisfactory on Monday may not be assumed to be so on Saturday.

WHAT CAN BE LEARNED FROM GRAND COULEE TEST DATA?

The only data secured from the tests are indicated by a, b and c; no "corrections" or slide-rule legerdemain can add anything. We cannot determine the weight of cement, sand or water. The following table gives an ideal measure of mixer efficiency which can readily be secured from the data:

Data from	% Lowest s/c*	$\frac{b-c}{b}$ —%			Mean Variation
		Front	Center	Back	
Patch	106	36.1	35.9	35.3	1.4%
Manual	105	33.0	33.5	33.6	0.6%

*As computed by the Bureau of Reclamation.

Mean variation was computed as the average of the differences (from the average) as a percentage of the average value; 3 tests are too few to furnish a reliable mean variation.

$\frac{b-c}{b}$ is the amount of sand, cement and silt washed through the 100 sieve, in terms of the total. This furnishes no clew to the relative quantities of cement or sand (neither does the method of the paper), but such a division is not necessary in order to secure a measure of mixer performance.

About 35 per cent of the dry material of the mortar was washed through the 100 sieve; we know that about 80 per cent of this is

cement, hence if the quantity of this material is constant for the different samples, it is practically certain that its composition will be constant also.

It will be noted that the mean variations in the 0-100 solids in the three samples average 1 per cent for the 2 tests. This represents a perfection in mixing which no one has a right to expect. No other operation in the manufacture, placing or testing of concrete approaches this accuracy. The writer interprets the tests as proof that the mix was uniform, and attributes the 1 per cent variation to errors of sampling and testing; a small margin when we consider the dozen steps where errors may enter.

The mean variation of 1 per cent should be contrasted with the factors 106 and 105 computed by the Bureau of Reclamation.

SAMPLES FOR CONCRETE TESTS

On the basis of standard practice an 18 x 36-in. cylinder is the smallest which could be used for compression tests of concrete containing 6-in. gravel. Such a cylinder weighs over 800 lb. and contains about 300 lb. of "mortar."

It should be apparent that there are almost unlimited possibilities for error in selecting a sample of 2 to 3 lb. from a 17,000-lb. batch of concrete. The efficiency of a 4-yd. mixer hinges on a test on less than 1 per cent of the quantity of mortar required for a compression test.

GRAND COULEE CONCRETE STRENGTH

28-day tests of 134 concrete cylinders made in September, show:

Average strength	5395 p.s.i.
Maximum strength	6400 p.s.i.
Minimum strength	3750 p.s.i.

The lowest strength was 28 per cent below the average; the spread between the min. and max. was 70 per cent.

Cement, water and aggregates were batched to the nearest 5 lb.; mixing conformed to the rigid requirements given in the paper. This no doubt represents the most perfect control ever attained.

The concrete tests were made by a skilled crew which had been making tests daily on this job for two years. We find no comment on the 70 per cent spread in concrete strength, while at the same time the mixers were held to a spread of 10 per cent, which appears to have been due entirely to variations within the range of the test methods.

If concrete strengths spread 70 per cent in a month, in spite of unprecedented control, how important is it that each little grain of cement and sand be in exactly the right place?

AUTHOR'S CLOSURE

The major part of Mr. Abrams' discussion, purporting to show that the mixer efficiency test is theoretically and fundamentally unsound, stands out in sharp contrast with the actual findings at Grand Coulee Dam and with information that the test has been used successfully to effect almost phenomenal correction of deficiencies in concrete mixing on a number of other Bureau projects. The paper describing this test was written to present a working method of solving a recognized field problem and not for the purpose of scientifically proving an abstract theory. If the article was not clear enough to enable the practical field man to make the test and demonstrate its efficacy, it is unfortunate, and it is hoped the following discussion will clarify all essential points.

During the time Mr. Abrams was employed at Coulee Dam by the contractors, Mason-Walsh-Atkinson-Kier Co., he was undoubtedly impressed with the *then* excellent performance of the mixing equipment, and apparently did not realize what a marked improvement had been made in two years through changes in batching sequences, and in designing entirely new blading for the mixers, both accomplished with the invaluable assistance of the mortar-mix tests.

Mr. Abrams implies in the first paragraph of his discussion that the Grand Coulee mixers gave excellent performance from the start. He continues with a criticism of the mixing requirements of the Bureau of Reclamation and concludes, "....however it should be stated that there *is* no rational basis for such a requirement, or for increasing mixing time with increase in mixer capacity." Such a conclusion assumes a knowledge that would close the door to further investigation. The Bureau engineers did not take this position, but by tests of a small scale model of the large mixers and by mathematical analysis of mixing action in small and large mixers accumulated abundant evidence supporting the need of longer mixing in large mixers. Furthermore, the contractor and the mixer and batcher manufacturers would not have spent weeks working in cooperation with the Bureau engineers in correcting and improving concrete mixing at Grand Coulee Dam if it had not been admittedly bad. Abundant physical and chemical data are on file to convince anyone who may be interested as to the facts.

The amount of mixing which a given batch of concrete receives in a concrete mixer, of a given design, is roughly proportional to the number of times that the concrete is "folded over" during the mixing operation. To cite a simple example: Assume a small hand-mixed batch using a wash basin and spoon. The amount of mixing is de-

pendent upon the number of times the spoon is forced through the mass, regardless of the total elapsed time. In other words, if the mass is beaten 20 times in 1 minute, it receives practically the same amount of mixing as if it were beaten 20 times in 2 minutes.

In a concrete mixer the blades take the place of the spoon and are forced through the mass as the drum revolves. Kinetic and mechanical considerations place rather definite limits on the maximum peripheral speed at which a mixer drum may be driven. The average peripheral speed for mixers from 1 to 4 cubic yards capacity used by one well known manufacturer is 268 feet per minute. With the peripheral speed more or less fixed, as the diameter of the mixing drum is increased, the revolutions per minute are correspondingly decreased and the length of mixing time increased to obtain the same amount of "folding" or mixing. If it is assumed that the number of blade-passes per revolution through the mass is independent of mixer capacity, then the length of mixing time required is roughly proportional to the diameter of the mixer drum.

Mr. Abrams states that "Mixing is one of a dozen steps in the manufacture and placing of concrete." The implication is that mixing is a relatively unimportant process. However, it is evident that the greatest care exercised in all the other steps could be largely nullified if the concrete is badly mixed. It is not uncommon to find regular mixing operations in which variations in S/C and W/C up to 100 per cent or more occur in samples from different portions of a batch. Generally the larger mixers are found to be the worst offenders. (See Table 2.)

The lack of definite data as to the action of large mixers only emphasizes the need of a mortar test such as described in order to compare the mixing action of different size mixers. Unsupported faith in the efficiency of all mixers, regardless of size, type, mix, consistency, or method of charging is certainly not justified.

Mr. Abrams' attempts to prove the mixer efficiency test unsound rest almost entirely on the single erroneous assumption that, "In spite of any amount of mixing of a given lot of sand, it is unlikely that 3 grab samples would show a spread of less than 1 per cent through the No. 100 mesh sieve." It should be noted that the assumed spread of 1 per cent in -100, which constitutes only 5 to 6 per cent of the total sand, is actually a variation in the -100 constituent of 16 to 20 per cent. The fact is that in hundreds of sand tests taken monthly at the aforesaid mixing plants, it is very rare to find a change of more than 1 per cent in the -100 even in an entire day's run. The plant often

runs for several days with less than 1 per cent variation in -100 samples. And this, it is to be noted, is before the sand goes into the mixer to be further blended. Table 1 shows the %-100 in sand tests during one month at one of the mixing plants. This is typical, and indicates how unfounded the above-quoted statement is.

Mr. Abrams states further: "The method of sampling was given scant attention in the paper." "This is a vital factor since mixer

TABLE 1

Tabulation of Mixing Plant sand grading tests for July, 1937, Coulee Dam, showing percentage of −100 in the sand. Grab samples from sand batcher taken each shift. Shown in consecutive order as taken from shift to shift.

Westmix

Date	Per Cent Sand Passing 100 Mesh														
July 1, 1937	5	5	6	6	5	5	5	5	5	5					
July 5, 1937	5	5	5	5	5	5	4	6	6	5	6	7	6	5	
July 10, 1937	6	6	6	7	6	6	6	6	6	6	5	5	7	5	5
July 15, 1937	7	6	6	7	6	6	6	6	6	6	5	7	6	6	
July 20, 1937	5	6	4	5	5	5	5	6	6	5	6	5	6	6	6
July 25, 1937	6	5	5	6	6	5	6	7	7	6	6	7	6	6	6
July 30, 1397	6	6	7	7	7										

Average 5.7%

TABLE 2

Mixer Efficiency Tests of Commercial Type Mixers of Various Sizes

Capacity of Mixer	Mixing Time Minutes	Sand-Cement Ratio % of Lowest S/C			Water-Cement Ratio % of Lowest W/C		
		Front	Center	Back	Front	Center	Back
4 cu. yd.	3	100	113	106	105	104	100
3 cu. yd.	2½	100	101	104	100	103	104
1 cu. yd.	1½	100	100	102	106	100	100
½ cu. yd.	1½	100	100	105	100	100	101
¾ cu. yd.	1½	101	100	101	100	100	103

efficiency hinged on one small sample. Grab samples taken from a mixer are notoriously erratic." Mr. Abrams in the above discussion has apparently missed the true purpose of the test, yet is verifying by his statement the very erratic condition these tests have disclosed in many mixers. This indicates the need of just such a test. This erratic condition has been eliminated to a large degree in the Coulee Dam mixing plants with the assistance of the mortar test. The purpose of the test is not to find what the *average* mix is. This is already known by closely controlled batch weights. The purpose of the mortar-mix test is to reveal when a batch is not well mixed, and small samples from widely different parts of the batch are most revealing. In other words the "grab" sample method is purposely used to discover the samples showing the greatest variations.

The quotation Mr. Abrams has taken from page 291 of the Bureau's 1938 Concrete Manual in regard to sampling has to do with taking samples for making slump tests or test cylinders, not for testing thoroughness of mixing. Had it not been recognized that mixing is likely

TABLE 3

TEST CYLINDER UNIFORMITY—MONTH OF JULY 1937

Strength in lb. per Sq. In.		% Variation Diff. x 100 / Min. Str.	Strength in lb. per Sq. In.		% Variation Diff. x 100 / Min. Str.
Cyl. No. 1	Cyl. No. 2		Cyl. No. 1	Cyl. No. 2	
5675	5535	2.53%	5260	5155	2.04%
5740	5830	1.57	4985	4820	3.42
5400	5480	1.48	5520	5400	2.22
4440	4785	7.78	5995	5695	5.27
5305	5535	4.33	6235	5490	13.55
6050	5705	6.14	5130	5040	1.79
5710	5630	1.42	5800	5465	6.12
5975	5935	0.67	5545	5445	1.84
5650	5055	11.80	5740	5730	0.17
5905	5660	4.33	5170	5170	0.00
5905	5730	3.05	4950	4790	3.34
5140	5250	2.14	5820	5550	4.87
5130	5520	7.60	5480	5340	2.62
5625	5410	3.98	6650	6435	3.34
5590	5500	1.64	5420	5320	1.88
5560	5340	4.12	5020	5010	0.20
5445	5340	1.97	5855	5855	0.00
5860	5500	6.55	4605	4490	2.56
5675	5675	0.00	5390	5370	0.37
5085	5280	3.84	5835	5825	0.17
5225	5340	2.20	4870	4680	4.06
5520	5750	4.17	5710	5615	1.69
5190	4985	4.12	6350	6240	1.76
5475	5245	4.38	5320	5270	0.95
4865	4775	1.88	5190	4915	5.59
5500	5280	4.17	5685	5520	2.99
5250	5215	0.67	5410	5305	1.98
4640	4640	0.00	5305	4750	11.68
5720	5340	7.12	5535	5340	3.65
5295	5020	5.48	5530	5110	8.25
5005	4680	6.93	5305	5005	6.00
6160	5905	2.62	5625	5625	0.00
5370	5260	2.09	5765	5625	2.49
5200	5145	1.07	5305	5280	0.47
5305	5295	0.19	5445	5385	1.11
5510	5320	3.57	5130	4835	6.11
5160	5055	2.08	5235	5145	1.75
5110	4970	2.82	4765	4490	6.12
4950	4720	4.87	6210	6155	0.89
5395	5320	1.41	5720	5640	1.42
5525	5200	6.25			
4720	4510	4.65			
5500	5480	0.37	Avg. % Variation		3.33%
5305	4780	10.98			
5110	5090	0.39	Mixing time 2 or 2¼ minutes.		

to be very imperfect and erratic, such a requirement for sampling would have been unnecessary.

Mr. Abrams further states: "Improved mixing seems to have been coincident with a change in the sampling." No such statement was made or intended to be inferred from the original paper, nor is it true. Not until the mixing action and operation was considered satisfactory was the plant allowed to go on a routine 2½-minute mixing schedule, and at that time the new procedure in sampling was adopted to prevent delay and restriction of the plant output.

TABLE 4

Variations in 6 x 12 Cylinder Strengths Due to Improper Mixing.

These tests were made early in the placing of concrete at Grand Coulee Dam and before any changes had been made in mixer blade design or charging sequence. Due to such erratic results as these, the mixer efficiency test was adopted as regular test procedure. Many weeks of design changes and tests followed so that the mixing time could be reduced from 5 minutes to 2½ minutes or less.

It will be noticed from the test results that high values of sand-cement ratios, made by mortar-mix methods, correspond to proportionally low compressive strengths and vice versa.

These tests were made after 2½ minutes of mixing.

	Sand-Cement Ratio			28-Day Strengths			Average Strength after 5 Min. Mixing
	Front	Center	Back	Front	Center	Back	
	3.1	2.3	1.2	3935	6235	7075*	——
% of low value....	258%	192%	100%	100%	158%	180%	
	4.4	3.3	2.5	2900	4775	6490	5902
% of low value....	176%	132%	100%	100%	165%	224%	
	4.1	2.9	2.1	3340	6085	6925	5985
% of low value....	195%	138%	100%	100%	182%	207%	
	2.7	3.3	2.7	6550	5385	6835	——
% of low value....	100%	122%	100%	122%	100%	127%	
						Average	5943

*Cylinder did not break at testing machine capacity.

TABLE 5

Comparative Tests of Two Methods of Taking Mortar Test Samples
Batching Varied to Develop Imperfect Mixing

Test		Samples Taken From Inside Mixer			Samples Taken When Mixer Dumped		
		Front	Center	Back	Front	Center	Back
A	% of lowest S/C.....	100%	105%	108%	105%	102%	100%
	% of lowest W/C.....	107%	100%	109%	103%	104%	100%
B	% of lowest S/C.....	106%	106%	100%	100%	103%	104%
	% of lowest W/C.....	102%	113%	100%	106%	106%	100%
C	% of lowest S/C.....	169%	156%	100%	182%	120%	100%
	% of lowest W/C.....	150%	132%	100%	175%	116%	100%

Mr. Abrams then concludes, "The writer has been unable to find one redeeming feature in the methods of computing or interpreting the results from the original data." It has been indicated that Mr. Abrams' difficulty comes from analyzing the data presented on the false basis that the -100 material in the sand cannot be kept within 15 per cent to 20 per cent variation. As the paper was written from the viewpoint of a practical field method, the fact that the method is working satisfactorily on a number of projects, not only to correct inherent defects of mixers and batching methods, but to maintain efficiency, seems sufficient justification for its use and publication. It was stated in the original article that values of S/C and W/C ratios are not necessarily the same as when batched, but it has been demonstrated that agreement in this respect is not to be expected and is not necessary for the purpose of the test, and the complicated procedure necessary to determine such values with certainty was

TABLE 6

Comparison of 36 Tests, Each Comprising Three Samples from the Same Batch by Mortar Test and by Chemical Analysis

Sample No.	Chem. Ana. *Largest %	Mortar Ana. Largest %	Diff.	Sample No.	Chem. Ana. Largest %	Mortar Ana. Largest %	Diff.
4A	104	102	—2	B-20	103	105	2
B6	100	102	2	22	102	102	0
7A	102	106	4	22A	100	102	2
10	107	102	—5	22C	101	102	1
9	104	105	—1	23	104	102	—2
8	102	104	2	23A	103	101	—2
10A	103	102	—1	23B	102	103	1
11	103	100	—3	B-23A	101	103	2
12	105	106	1	23C	103	101	—2
12E	101	102	1	23E	103	103	0
12F	101	103	2	24	101	102	1
12G	106	104	—2	24A	101	104	3
12H	100	102	2	24B	105	103	—2
12I	102	102	0	24F	101	103	2
13	105	108	3	24G	101	104	3
B-13	102	105	3	24H	102	101	—1
B-13A	102	103	1	24I	103	105	2
13D	102	103	1				
17A	101	101	0				

Average Variation	1.7%
Maximum Variation	5%
Minimum Variation	0%

*Largest % variation of 3 samples. Lowest equals 100%.

accordingly eliminated. The limits established were based on actual results determined from the local plant after improvements in blades and batching sequences had been made, and it was found that variations within the 10 per cent spread could readily be maintained for indefinite periods with only occasional lapses. Frequently such lapses can be traced to definite irregularities in batching or mixing, and are promptly corrected.

In the second paragraph of Mr. Abrams' discussion under the heading "Computations based on a Fallacy," the question is asked, "How can one fine material in a batch be uniformly mixed without the other fine materials being uniform also?" Evidently he is intimating that if the cement is not uniformly distributed throughout the batch the fine sand could not be. The obvious answer to this is that the various sizes of sand are well mixed before entering the mixer but the cement is batched independently.

It has been found in all tests tried on many kinds of mixers, that continued mixing is reflected in improved mortar tests until a variation in S/C and W/C of less than 5 per cent is reached. Usually, with small mixers, 1 per cent to 3 per cent variation limits may be reached by continued mixing. (See Tables 9 and 2.) Obviously, absolute correction factors would be impossible of predetermination on account of progressive grinding action, as well as other variables, but the fact that long mixing even with a poor mixer will finally produce concrete testing up to the requirements, and furthermore the fact that corrections in batching and blading made with the assistance of the

TABLE 7

Showing both by Mortar Test and by Chemical Analysis the uniformity of 20 to 30 per cent by volume of silicious admixture replacing cement in mass concrete. Both added at mixer, unblended. Four-yard mixers. Mixing time usually 2½ minutes.

Batch No.	Actual % Cement in Cement Pozzolan Combination	Average % Cement by Chemical Analysis in Batch	Maximum Variation between 3 Samples in batch (Chem. Analysis)	Maximum Variation between Batched % Cem. and % Found by Chem. Analysis	Mortar Test S/C Maximum Variation
B-14	83.1	85.0	2%	1.9	4%
B-2	83.1	82.0	0	1.1—	3
B-4	81.7	83.7	2	2.0	5
B-4A	81.7	77.3	1	4.4—	7
B-4B	81.7	80.7	1	1.0—	3
B-10	82.3	85.3	2	2.0	3
B-10A	82.3	82.0	2	0.3—	2
B-23	83.9	86.7	1	2.8	3
B-23A	83.9	82.3	1	1.6—	6
2A	75.0	73.3	2	1.7—	5
2B	75.0	75.3	1	0.3	3
7	75.0	76.0	2	1.0	10
7A	75.0	73.7	1	1.3—	3
8	72.5	79.3	2	6.8	4
8A	72.5	78.3	2	5.8	3
2	75.0	75.7	1	0.7	3
4	72.5	75.3	5	2.8	2
4A	72.5	76.7	3	4.2	1
4B	72.5	72.7	3	0.2	7
10	72.5	74.7	5	2.2	2
10A	72.5	75.0	2	2.5	3
10B	72.5	72.0	2	0.5—	1
9	74.0	76.7	3	2.7	7
2W	78.4	80.7	3	2.3	37
23	75.0	76.7	3	1.7	4
23A	75.0	75.7	2	0.7	1
23B	75.0	81.7	1	6.7	1
23C	75.0	72.7	2	2.3—	4
23D	75.0	77.0	2	2.0	4
23E	75.0	75.3	2	0.3	22
5	74.0	77.3	4	2.3	1
		Average	2	2.2	5%
		Maximum	5	6.7	37
		Minimum	0	0.2	1

mortar test will often enable a formerly poor mixer to accomplish the same satisfactory mixing in a fraction of its original time is proof of the value of the method.

The mortar test was developed from the basis that when a batch of concrete is thoroughly mixed, all ingredients (sand, silt, water, and cement) will be uniformly distributed throughout the mortar, and if correction factors used are slightly in error all samples should be affected alike, so results will still be comparative. On the other hand, if mixing is not thorough, correction factors that are not approximately correct may give erroneous indications. (See Table 8.)

Mr. Abrams remarks that it is not clear why a mixer that performs satisfactorily at one time should not do so at another time. To cover this point, test data are shown which indicate results of variations in charging, blading, or mixing time. (See Tables 4, 5, 9, and 10.)

Mr. Abrams says that data, a, b, and c are all that can be secured from the test. If that were true, the test would be of little value.

TABLE 8

Computations on Hypothetical (Synthetic) Samples showing the results when correct factors are used, and trends when incorrect factors (f) are chosen: Percentages shown are based on lowest S/C or W/C. Theoretical mix—(1-3.0, W/C 0.6).

Sample Size	%—100 in Sand		800		1200		1600	
Computed results in:	Actual	Used (f)	S/C W/C	% %	S/C W/C	% %	S/C W/C	% %
(a) Mixed to uniform condition	10%	10%	3.00	100			3.00	100
			.60	100			.60	100
Mixed to uniform condition	10%	5%	2.55	100			2.55	100
			.53	100			.53	100
Mixed to uniform condition	10%	15%	3.82	100			3.82	100
			.71	100			.71	100
(b) Uniform mix except —100 varied 10% & 8% & 10%	10, 8, 10%	10%	3.00	100	3.28	109	3.00	100
			.60	100	.64	106	.60	100
Uniform mix except —100 varied 10% & 8% & 10%	10, 8, 10%	5%	2.48	100	2.67	108	2.48	100
			.53	100	.56	105	.53	100
Uniform mix except —100 varied 10% & 8% & 10%	10, 8, 10%	15%	3.82	100	4.24	111	3.82	100
			.71	100	.77	108	.71	100
(c) S/C varies 1:2.5 & 1:3.0 & 1:2.5 same proportion water each sample	10%	10%	2.50	100	3.00	120	2.50	100
			.53	100	.60	114	.53	100
(d) Uniform mix except W/C varies 0.5, 0.6 & 0.5	10%	10%	3.00	100	3.00	100	3.00	100
			.50	100	.60	120	.50	100
(e) Uniform mix but Sp. Gr. of cement varies 3.00, 3.15, and 3.00	10%	10%	3.02	102	2.95	100	3.02	102
			.61	107	.57	100	.61	107
(f) Uniform mix but Sp. Gr. of sand varies 2.70, 2.75 and 2.70	10%	10%	2.99	100	3.02	101	2.99	100
			.61	105	.58	100	.61	105

TABLE 9

De-graded Batching Mixer Tests

All tests were made at MWAK Westmix Plant.

Tests had been made after extensive blade changes and revisions in charging sequence. The normal mixing time previous to these tests was 2 min. This shorter mixing time had allowed MWAX to make many record placements. Samples were removed from front and back of mixer at various intervals.

The following tests show the effect of improper charging of material and the time necessary to get good mixing.

Mixing Time Min.	Test No. 1 (Control) Regular Charging Mix: 6AQRR, Slump 2" Gross W/C 0.64		Test No. 2 (Batch A) Degraded Charging Mix: 6AQRR, Slump 1½" Gross W/C 0.63		Test No. 3 (Batch B) Degraded Charging Mix: 6AQRR, Slump 1½" Gross W/C 0.60		Test No. 4 (Batch C) Degraded Charging Mix: 6AQRR, Slump 2" Gross W/C 0.59	
	% Varia. in S/C	% Varia. in W/C	% Varia. in S/C	% Varia. in W/C	% Varia. in S/C	% Varia. in W/C	% Varia. in S/C	% Varia. in W/C
½	42%	190%	179%	72%	314%	28%	—	—
1	28	4	21	213	16	12	—	—
1½	5	3	1	15	23	13	84	70
2	0	1	3	24	18	9	0	38
2½	—	—	17	5	12	3	7	6
3			4	7	10	63	1	1
3½			—	—	5	0	—	—
4					1	6		

TABLE 10

Bureau of Reclamation — Columbia Basin Project

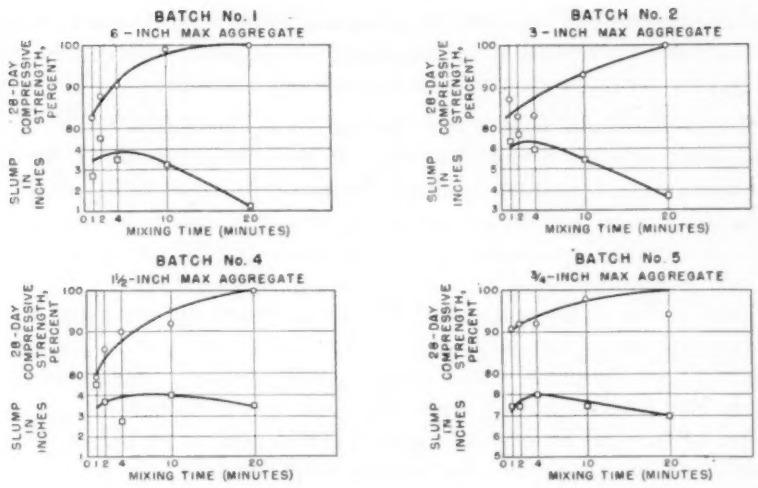
Mixer Efficiency Results

East Mixing Plant — Date Nov. 4. 1938 to Nov. 10. 1938

Date	Shift	North Mixer			East Mixer			South Mixer			West Mixer		
		Maximum		Hour Mixed	Maximum		Hour Mixed	Maximum		Hour Mixed	Maximum		Hour Mixed
		S/C	W/C		S/C	W/C		S/C	W/C		S/C	W/C	
4	N	101	102	11:05	103	113*	2:10	102	109	3:25	103	103	5:25
	D	104	109	7:10	104	108	9:25	103	100	11:05	105	107	1:05
	S	105	104	3:10	111†	110	5:15	104	103	6:00	101	104	8:00
5	N	101	101	11:30	105	100	2:15	105	103	3:20	104	103	5:20
	D	100	103	7:10	106	103	9:15	103	104	11:10	105	103	1:15
	S	106	106	5:05	101	102	8:05	109	105	3:30	104	104	5:05
6	N	101	103	11:05	101	104	1:15	102	102	3:15	102	102	4:15
	D	108	106	7:10	104	101	9:05	106	101	11:10	101	101	1:10
	S	105	104	7:10	101	104		109	105	6:10	107	104	8:15
7	N	102	101	11:10	102	102	2:15	102	101	3:20	102	101	4:25
	D	103	105	7:10	103	106	9:05	104	105	11:05	102	102	1:00
	S	105	105	3:30	106	103	5:20	105	103	6:10	102	104	8:05
8	N	101	103	11:10	101	104	1:15	104	106	2:15	101	102	4:15
	D	102	104	7:15	101	101	9:00	115‡	106	11:45	103	118‡	
	S	103	102	5:00	101	102	6:00	100	102	8:10	105	105	3:05
9	N	100	101	11:15	104	102	2:10	100	103	3:10	103	102	4:25
	D	104	106	7:10	105	103	9:00	100	102	11:00	102	103	1:05
	S	104	103	5:05	106	105	6:00	104	103	3:20	105	106	8:05
10	N	107	108	11:20	100	103	2:25	107	106		102	106	
	D	104	102	7:05	106	105	9:05	106	106	11:05	101	104	1:05
	S	108	109	3:10	107	102	5:35	104	104	6:45	106	109	

*East Mixer—Batch stood ten minutes in dead mixer.
†East Mixer—Cement hopper opened late.
‡South and West Mixers—1½" Max. Agg. batching sequence accidentally used.
Note: All Mortar tests shown made on 6" Max. Agg. Mixes.

The statement of Mr. Abrams presupposes that nothing is known about the fines in the sand, or whether they are mixed uniformly throughout the batch. When the test was being developed a large number of mortar samples were tested by chemical methods, and found in very close correlation with mixer mortar tests. (See Table 6.) It is also true that before the sand goes into the mixers at Coulee Dam, a variation of over 1 per cent in the -100 is very rare. (See Table 1.) This condition is undoubtedly improved still more in mixing because other chemical and mortar tests indicate that not only the cement, but siliceous admixtures amounting to several times the amount of



NOTE: Strengths are plotted as percentages of strengths for 20 minute mixing

FIG. 1—EFFECT OF INCREASED MIXING TIME ON CONCRETE STRENGTH AND SLUMP

Strengths are plotted as percentages of strengths for 20 minute mixing.

-100 in the sand, batched independently, can be mixed to a similar uniformity in the usual mixing time. (See Table 7.)

With the $\frac{b-c}{b}$ relation shown by Mr. Abrams, no indication is given of the water distribution, which, thanks to Mr. Abrams' former work, is now recognized as vital. It was frequently found in the large mixers when first tested that sand and cement proportions were satisfactory, but water distribution varied tremendously. The complete method reveals this condition sufficiently well. (See Table 9.)

Under Mr. Abrams' discussion of "Samples for Concrete Tests" he intimates that samples for mortar tests should be 800 lbs., similar in size to those for compression tests. Here again Mr. Abrams seems to misunderstand the purpose of the tests and their application. Obviously if the materials were carefully batched, and the entire batch taken as a sample, there would be no indication of nonuniformity that might be readily disclosed by small samples, as actually used in the tests. The mortar test assumes that with good mixing, a sample of any given size will be as intimately mixed in a large mixer as in a small mixer. There is no distinction made in design or in specifications between concrete mixed in small mixers and that mixed in large mixers. That the 10 per cent variation requirement is not extreme or imprac-

tical is indicated by the results shown in Tables 2, 6, 7, and 10 where, by both mortar tests and actual chemical analyses, such requirement for mixing action is shown to be consistently practical.

The reference to 70 per cent spread in concrete strengths during the month indicates a wide difference in the view points of Mr. Abrams and the author. Because cements from the different mills, and even from different runs of clinker from the same mill, differ 40 per cent or more in strength, and because variations in aggregate grading are not entirely eliminated, and because these and other factors still operate to make three cylinders out of 134 drop below 80 per cent of the average strength, the author does not feel that there is no use in attempting to improve other steps in the process. The manner in which Mr. Abrams states the comparison is very misleading. As a matter of fact the mortar test spread is not held to 10 per cent variation any more than strengths are, for while the aim is to keep within certain limits in both mortar mix and cylinder strength uniformity, both occasionally overstep the limit. In this connection it may be of interest that the average spread in compressive strength between the daily cylinders from the same batch, which is a better indication of effective concrete testing control, is very low. (See Table 3.)

Under the heading "Tests Misinterpreted"—Mr. Abrams states, "The evidence available indicates that the concrete was probably grossly over-mixed when the samples were taken after two minutes' mixing. There is nothing in the report to show that the concrete was not sufficiently mixed at 1 minute."

The author states in the first paragraph of the paper under discussion, that tests showed 250 per cent variation in the S/C and W/C in different parts of the mixer after 2½ minutes' mixing. In the author's opinion, neither the Bureau engineers, the contractor, nor the mixer manufacturer ever considered that this was due to over mixing, or that the concrete was sufficiently mixed at less than half that length of time. Mr. Abrams does not state what the "evidence" was, but we are showing in Fig. 1 some typical charts from a large series run with the 4-yard mixers, after they were mixing satisfactorily, which indicate improvements beyond the 2-minute mixing period in both compressive strength and slump.

According to Mr. Abrams, "The author has penalized the mixers for errors concealed in sampling, and which could probably not be eliminated by any practicable amount of mixing." It will be noted from the original paper that tests were considered satisfactory unless the variation in the three samples was over 10 per cent, and even

then no "penalty" was imposed until a second test was run confirming the poor result. That a small amount of additional mixing does eliminate such results is shown by Table 9.

Table 9 also shows the effects of continued mixing where batching conditions were such that normal mixing time was insufficient.

SUMMARY

Apparently Mr. Abrams' discussion is based on the belief that concrete mixers are equally efficient from hour to hour, day to day, and year to year regardless of variation in the mixers themselves or in charging conditions or changes in mix or mix consistency, and on misinterpretations relative to the purpose and method of making the mixer efficiency tests as reviewed below:

1. That the mixer performance at Grand Coulee was good from the start and was not improved by changes made. This is untrue.

2. That mixing efficiency should not be expected beyond the variation of 15 to 20 per cent in uniformity of distribution of the -100 sand in a batch of concrete. Better efficiencies than this are consistently maintained at Coulee Dam.

3. That a large mixer will mix a large batch thoroughly as quickly as a small mixer will mix a small batch well. This is an established fallacy.

4. That the purpose of the test was to find out exactly how much cement, water, fine sand, and coarse sand was contained in representative samples taken from the mixer, rather than to determine the relative proportions of cement, water, and sand in samples selected to show the greatest variations in these constituents. This indicates a misunderstanding of the purpose of the test.

5. That concrete samples taken to test uniformity of mixing should be taken as specified in the Bureau's Concrete Manual, page 291, for taking samples for slump or strength tests. Such procedure would defeat the purpose of the mortar mix test.

6. That because some of the factors could be varied without affecting the comparative results, the whole procedure was useless. Instead, it is actually safer.

7. That the S/C and W/C should be exact to be of any value. This was shown to be untrue.

8. That results obtained are known to be grossly in error. Results are shown to be sufficiently accurate for the purpose intended.

9. That it is a practical impossibility to so thoroughly mix a given lot of sand that three grab samples would show a spread of less than

1 per cent through the 100 mesh sieve. Thousands of tests at Coulee Dam prove this statement false.

10. Mr. Abrams repeatedly refers to the S/C and W/C as "items known to be incorrect." Although they are items which may not be absolutely correct, they are suitable for the purpose of the test.

In closing it is hoped that the interest developed by the discussion of this paper will prove to be of real value, leading to further improvements in the method or to a better method to accomplish the same purpose.

BY ARTHUR RUETTGERS*

The name of Duff A. Abrams is familiar to every student of concrete by reason of the lasting contributions to the fund of concrete knowledge with which his name is associated; consequently, his discussion of Mr. Patch's paper is likely to be widely read. It is most disappointing, therefore, to find that the discussion contains a scathing denunciation of the procedure developed at Grand Coulee Dam for testing the efficiency of concrete mixers and an implication that the Bureau of Reclamation was unwarranted in accepting the procedure for general use. Unpleasant though it is to take issue with Mr. Abrams, to do otherwise is to submit to criticism of a destructive character.

The Bureau did not act hastily in endorsing the mixer-efficiency test by incorporating it in the Concrete Manual and in several recent construction specifications. The development work preceding use of the test at Grand Coulee Dam dates back to 1935; the test was subsequently tried out with marked success on the mixers used in the construction of four other major Bureau dams; it was incorporated in specifications dated December 10, 1937, for Grand Coulee High Dam; and it was first published in the Concrete Manual in July 1938. The following account of actual experiences and the practical results obtained in applying the mixer-efficiency test at the four dams, as mentioned, serves to demonstrate its utility.

Dam A (October 1937)

Reason for tests. Harsh concrete. Widely varying strength of test cylinders. Visible lack of uniform mixing.

Results of preliminary tests. Mixing efficiency for mixer No. 1, 29 per cent; for mixer No. 2, 48 per cent; average for two mixers, 39 per cent.

Principal alterations. Change in design of blades and waterspout. Improved control of batching and charging operations.

Results of final tests. Mixing efficiency for mixer No. 1, 99 per cent; for mixer No. 2, 95 per cent; average for two mixers, 97 per cent.

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Dam B

Reason for tests. Average mixing efficiency of 92 per cent in January 1938 for four mixers dropped to 46 per cent in November 1938. Increased mixing time showed improvement.

Principal alterations. Replaced worn mixer blades.

Results of final tests. Mixing efficiency with regular mixing period, 93 per cent.

Dam C (March to May 1938)

Reason for tests. Mixer replaced.

Results of preliminary tests. Mixing efficiency, 67 per cent.

Principal alterations. Clearance between blades and liner increased. Introduction of water altered.

Results of final tests. Mixing efficiency, 90 per cent.

Dam D (3-inch maximum aggregate, May 1937)

Reason for tests. Visible nonuniformity in mixing. Pronounced segregation in discharging.

Results of preliminary tests. Mixing efficiency, 38 per cent.

Principal alterations. Releveled mixer.

Results of final tests. Mixing efficiency, 98 per cent.

	Front	Center	Back
W/C	0.45	0.46	0.45
S/C	2.34	2.38	2.37
7-day strength	2,577	2,566	2,547

Dam D (6-inch maximum aggregate, March 1938)

Reason for tests. Starting production of mass concrete containing cobbles. Noticeable variation in consistency in mixer.

Results of preliminary tests. Mixing efficiency, 73 per cent.

Principal alterations. Improved regulation of manual batching and charging operations.

Results of final tests. Mixing efficiency elevated to 97 per cent.

In addition to the above, the mixer-efficiency test has been employed on various other Bureau projects during the past year, and, in no instance, to the writer's knowledge, has there been reason to question its efficacy.

Mr. Abrams states that the author's computations for determining the spread in sand-cement ratio of three mortar test samples are based on a fallacy, in that it is improper to assume that the sand in all three samples contains the same percentage of material passing the No. 100 screen. He then proceeds to explain how a small variation between samples in the percentage of minus 100 sand may greatly affect the computed spread in sand-cement ratio, and, finally, he presents a method of stating results which he states "gives an ideal measure of mixer efficiency." However, field tests on several projects indicate that Mr. Abrams is in error in assuming that there is likely to be a

large amount of relative variation in the minus 100 sand. In any event, his "ideal measure" would likewise be affected by such variation, and, moreover, it does not measure separately the variations in the sand-cement and water-cement ratios. Incidentally, there appear to be six arithmetical errors in the ten figures which he presents in illustrating his index.

Mr. Abrams takes exception to the index used as a measure of the efficiency of the mixer (termed "efficiency factor" by the author) and would prefer the use of the standard deviation or some similar function for measuring uniformity. As a matter of fact, the author's index, made up of the average of the percentages of samples coming within three specified variations ("range" as used in statistics), is closely related to the standard deviation within the values for which it is significant. It has the advantage of being easily computed and readily understood by the men concerned. It is of interest that the "ten per cent limit for spread", found by field experience to be indicative of the need for correction in some phase of the mixing or charging process, corresponds approximately to the value "three times standard deviation" often used for the control of industrial processes.

With regard to Bureau specifications for mixing time, which generally require a minimum of 1½ minutes for mixers of two yards' capacity or less and a sliding-scale increase in mixing time for mixers of larger capacity, Mr. Abrams states:

> There is no doubt of the right of the Bureau to demand that a 4-yard batch be mixed 2½ minutes; however, it should be stated that there is no rational basis for such a requirement or for increasing mixing time with increase in mixer capacity.

And later on:

> The evidence available indicates that the concrete was probably grossly overmixed when the samples were taken after two minutes mixing. There is nothing in the report to show that the concrete was not sufficiently mixed at one minute.

Mr. Abrams presents no data or references to support these statements, although there is an excellent paper by Mr. Abrams on "Effect of Time of Mixing on the Strength of Concrete", reporting the results of tests made about 20 years ago. However, these tests were made under laboratory conditions and using a mixer of only 3½-cu. ft. capacity, whereas the present discussion relates to mixers of about 30 times this capacity, operating under field conditions.

Bureau requirements on mixing time are the outgrowth of experimental work and practical experience over an extended period and are aimed not only to insure intimate mixture and uniform distribution of the materials but to enhance the workability of the concrete. They also take cognizance of the many factors that may operate to delay

the fulfillment of these objectives, such as differences in types and makes of mixers, condition of the mixer, charging sequence, characteristics of the cement and aggregates, number of batched materials to be combined, and consistency of the concrete. Thanks to the development of the mixer-efficiency test, the Bureau has been enabled of late to provide a more flexible specification requirement with respect to minimum mixing time by stipulating that the minimum period specified may be shortened in the event the results of the mixer-efficiency test show it to be justified. In this connection, it is of interest that, during the construction of Boulder Dam, a request for reduction in the 2½-minute minimum mixing time for the 4-yard mixers was referred to a board of consulting engineers and was denied, the expression of the board being as follows:

> There is general information, based on both experiment and field experience, that slump, workability, and strength increase in general up to a certain point, with increase in mixing time. These results, however, depend on the character of the cement, the size of the mixer, and other special conditions. There seems to be available, however, no data based either on experiment or field experience with the low-heat cement now in use, which would serve as a basis for judgment on these points. The Board is further somewhat apprehensive that a reduction of the time of mix below 2½ minutes now prescribed might result in some loss in the assurance of regularity and uniformity in the batches which this period secures. There is also no assurance against some loss in desirable physical properties. Under these conditions and with the information at present available, there seems no reason for recommending a shorter time of mix.

It is of further interest that, two years prior to the date of the board report containing the above quotation, tests made with the 4-yard mixers at Owyhee Dam indicated that workability as reported at the point of placing was improved by mixing beyond the prescribed minimum period of 1½ minutes.

The graphic records of concrete consistency meters required by the Bureau in the construction of major concrete dams provide information which, in the opinion of the writer, is positive evidence in support of the position that one minute of mixing in large mixers is inadequate and that longer mixing is required as the mixer capacity increases. A study of these records for four dams shows that the time required for the consistency-meter pen to approach a state of rest ordinarily ranges from 0.7 to 1.7 minutes for 4-yard mixers and from 0.7 to 1.1 minutes for 2 yard mixers. An additional lapse of time is, of course, required before the operator can be assured that the pen has settled down to a point where the graph furnishes an index of the consistency of the concrete. Also, whenever a batch is found to be too dry and tempering water is added, there is a further consumption of time of at least half a minute.

It is apparent from the following tabulation of mixing times specified or used on a number of large dams with various sizes of mixers that Bureau practice in this respect corresponds with that of others.

Name of Dam	State	Year Completed	Mixing Time in Minutes for Mixer Capacities of			
			1 Yd.	2 Yd.	3 Yd.	4 Yd.
Dams built by Bureau of Reclamation						
Elephant Butte	New Mexico	1916			1½	
Black Canyon	Idaho	1924	1½			
American Falls	Idaho	1927		1¼		
Stony Gorge	California	1928		*1½		
Gibson	Montana	1930		1¼		
Deadwood	Idaho	1931		2		
Owyhee	Idaho	1932				1½
Boulder	Arizona-Nevada	1935				2½
Grand Coulee	Washington	——				2½
Marshall Ford	Texas	1939		2		
*1½-yd. mixer.						
Dams built by other agencies:						
Wilson	Alabama	1924		2		2½
Melones	California	1926		1½		
Exchequer	California	1926		1½		
Pardee	California	1929		1½		
Pacoima	California	1929	1½			
Bull Run	Oregon	1929		2		
Waterville	North Carolina	1930		2		
Tujunga No. 1	California	1931		2		
Morris (Pine Canyon)	California	1934				3
Madden	Canal Zone	1935		2		
Roy B. Inks	Texas	1937		2		
Norris	Tennessee	1937			2½	
Wheeler	Alabama	1937		1½		
Tygart River	West Virginia	1937			2	
Bonneville	Oregon	1937			2½	
Flathead Hydroelect. Devel.	Montana	1938		2½		
Pickwick Landing	Tennessee	1938		1½		
Claytor Hydroelect. Devel.	Virginia	——		2		

While it will doubtless be necessary for some time to come to continue to incorporate a minimum mixing-time requirement in construction specifications, if for no other reason than to provide approximate information for prospective bidders, an assured uniformity of the mixer product with a minimum period of mixing is, after all, the primary aim. The writer is convinced that the Grand Coulee Dam mixer-efficiency test is decidedly a forward and practical step in providing the needed assurance and in stimulating future improvements in the design and performance of mixing and charging equipment.

The importance of this subject prompts extension of the discussion period to July 20 for the September Supplement.

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Discussion of a paper by O. G. Patch:

Mixer Efficiency or Mortar-Mix Tests*

BY DUFF A. ABRAMS† AND BY THE AUTHOR

INTRODUCTION

The writer is taking advantage of the opportunity offered by the Institute to present a further discussion of the above paper; this is in a measure supplemental to my discussion in the June JOURNAL, but is based largely on the statements of Messrs. Patch and Ruettgers and on the new tests submitted by them.

These writers devoted 17 pages to my criticisms of the Bureau of Reclamation mixer test. It is notable that neither chose to devote a few lines to the important point at issue: namely, that the *after-mixing* percentage of 0-100 sand in a batch was assumed to be uniform to four significant figures, as a result of an error in elementary arithmetic.

Instead of discussing the above they go at great length into the entirely irrelevant question of the *before-mixing* uniformity of the sand; a subject that had not heretofore been mentioned, either in the original paper or in my discussion.

My discussion also called attention to the over-mixing of concrete at Grand Coulee Dam and to the needless repetition of tests on mixers which were known to be performing satisfactorily.

Mr. Patch declines to furnish information on the method of securing mortar samples for the mixer test. The method is of utmost importance in view of the new tables in which sampling methods are notably absent.

*JOURNAL Amer. Concrete Inst., Jan. 1939, *Proceedings* Vol. 35, p. 173. See also June JOURNAL, p. 180-1 to 180-24, for discussion by Duff A. Abrams, O. G. Patch and Arthur Ruettgers. As provided by the A. C. I. Board of Direction (see June p. 180-24) supplemental discussion is published.

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RELATIVITY IN LARGE DAMS

Grand Coulee Dam will contain 10,500,000 cu. yd. of concrete. The dam is built in blocks 5 x 50 x 50 ft., each of which contains 463 cu. yd.; there will be about 20,000 such blocks. The smallest concrete unit is a 4-yd. batch; there will be 2,600,000 batches; each batch is welded by vibration to 3 to 10 similar batches. Minor variations from the average in the properties of the concrete within these units can have no determinable effect on the behavior of the dam. Of course special consideration must be given to concrete at or near exposed surfaces.

The above facts should be taken into account in weighing the relative importance of concrete mixing. There is nothing to suggest the need for striving toward a uniformity of 1 per cent in each pint-cup-full of mortar in a batch. The absurdity of this practice becomes apparent when we find that the strengths of different batches spread 70 per cent in a single month.

A CASE OF SIGNIFICANT FIGURES

Mr. Patch states:

> The paper . . . was written to present a working method of solving a recognized field problem and not for the purpose of scientifically proving an abstract theory.

The reference to "abstract theory" is too deep for me; however there are certain principles of elementary arithmetic which we must all follow, whether they are applied to a field problem or an abstract theory.

In order to compute water- and sand-cement ratios Mr. Patch assumed a fixed quantity of sand passing the 100-mesh sieve, *accurate to four significant figures.* For this to be true the batch must be thoroughly mixed before the samples are taken. A proper interpretation of the data shows that in the routine operation the concrete was mixed to a uniformity of 1 per cent when the samples were taken at the end of 2 minutes mixing.

The fallacy of this test is concealed in a case of "significant figures." In "correcting" for 8 per cent of 0-100 sand in the batch, a factor .92 was used. Since the sieve analysis is not accurate to less than 1 per cent of the total sand, .92 must include values ranging from .915 to .925; this is what Mr. Patch intended to assume. What he actually used was .9200, which means the range .91995 to .92005; all mixer tests are based on the assumption that the 0-100 sand in the batch was *uniformly distributed* within these limits. Little more need be said to show the absurdity of this assumption.

.92 and .9200 are the same on a slide-rule or a calculating machine, but they are quite different when applied to physical measurements; the first indicates that the experimental data are accurate to 2 significant figures, the second, to 4 significant figures.

The fallacy of the BR mixer test hinges on an error in elementary arithmetic; it is impossible to introduce into a computation a factor having two significant figures and come out with a result which is accurate to four figures. The BR method requires that computations be carried to four figures; this represents an accuracy 100 times greater than is possible from the assumption of 8 per cent 0-100 sand or from tests made with a No. 100 sieve.

Mr. Patch computes "original" weights of cement, sand and water (as they were termed in the January JOURNAL) which are known to be wrong. As a general principle it is not sound engineering to divide one wrong number by another wrong number and expect to get a right answer; this is what was done in computing s/c and w/c. The indication of these wrong values is then greatly exaggerated by computing a spread on the basis of the lowest, giving no weight whatever to the middle value or to the average.

DISCUSSION OF NEW TABLES

Mr. Patch has submitted 10 tables and one diagram of new tests; it seems desirable to present a few notes on these.

Table 1. Percentage 0-100 Sand at Grand Coulee. Tests are discussed below under "0-100 Sand Before and After Mixing."

Table 2. Efficiency of Commercial Mixers. Important data omitted: method of sampling; original mortar test weights a, b, c; type of mixers.

These tests show spreads ranging from 1 to 13 per cent; only one value is higher than 6 per cent; average 4.4 per cent. My computations showed that the *mean variation* in the concrete in the mixers averaged about $\frac{1}{5}$ of the "spread" in s/c computed by Mr. Patch. The highest average maximum variation in any batch would be of the order of 2 per cent (4-yd. mixer; s/c, 13 per cent; w/c, 5 per cent) and the general average 1 per cent or less. I conclude that the 4-yd. batch was sufficiently mixed (according to the BR criterion) and that the other batches were over-mixed long before the samples were taken. Similar results are shown by Tables 6 and 10.

It is strange that a 4-yd. batch mixed 3 min. was used in this table, when there are 8 mixers of this size at Grand Coulee which have turned out thousands of batches with mixing errors of 0 to 3 per cent after only 2 min. mixing.

Table 3. Cylinder Uniformity. The original paper gave a summary of concrete strengths at Grand Coulee during Sept., 1937, and showed a spread of 70 per cent between low and high. Table 3 shows for July, 1937:

Minimum single cylinder	4440 p.s.i.
Maximum single cylinder	6650 p.s.i.
Spread	50%
Minimum 2 cylinders from batch	4550 p.s.i.
Maximum 2 cylinders from batch	6540 p.s.i.
Spread	43%

The highest and lowest average strengths came within 24 hours, as indicated by only three values between them. Computations based on the averages of concrete in three successive samples in two instances show mean variations of 11 per cent.

Mr. Patch focuses attention on the uniformity in twin cylinders from a batch; this shows consistent testing, but is of no importance whatever when the strength of concrete in different batches spreads 50 to 70 per cent. We note that twin cylinders from the same batch show spreads up to 13½ per cent.

This table furnishes another example of erroneously carrying results to four figures, when the data scarcely justify three. Unavoidable errors in total load due to machine error and reading will be somewhere in the range 1000 to 2000 lb.; or a unit stress in a 6-in. cylinder of 35 to 70 p.s.i.

The tendency to bunch total loads at even thousands is clearly shown; for example, 5305 occurs 7 times; this is too many to be accidental; however we note that this corresponds to a total load of 150,000 lb. We then observe that 5340 occurs 6 times; this corresponds to a total load of 151,000 lb.

The true accuracy of such tests would be expressed by recording the strengths to the nearest 50, instead of the nearest 5 p.s.i. We are certainly going far enough when we express the values to the nearest 10 p.s.i.; this has been the custom of many testing organizations since the writer introduced the practice about 20 years ago.

Table 4. Variations Due to Improper Mixing. Important data omitted; original mortar test weights a, b, c; significance of items in first column; method of sampling; w/c values; number of concrete tests.

Probably grab samples were taken after stopping the mixer. The writer has already called attention to the erratic nature of such samples; in later tests Mr. Patch used a different method.

The mean variation in concrete strength in the last test is 10 per cent; whereas 11 per cent was found in three successive samples tested in the regular run 1 or 2 years later. (See under Table 3.)

The 30 per cent spread in average 28-day strengths (4720 to 6160 p.s.i.) is not explained for batches presumably alike and all mixed for 2½ minutes. We must conclude that the 3 concrete samples did not represent the average of the batches.

It is unlikely that such variations would be found for the same batches in the dam. The BR "Manual" requires under "Concrete Sampling" (p. 291)—"If practicable, samples shall be taken from the concrete in the forms or from a convenient point just before it reaches the forms." There is no reason why samples should not be taken from the forms at Grand Coulee, except that it is more convenient for the laboratory forces to take them at the mixer.

The 4-yd. batches at Grand Coulee were turned over two to three times after the test samples were taken and before the concrete was in place; this has an important influence in improving the uniformity of the concrete in a given batch.

Table 5. Two Methods of Taking Samples. Important data omitted: original mortar test weights a, b, c; time of mixing; identity of A, B, C; method of securing samples.

The table is given in factors which the writer has shown to be erroneous.

There is little point in deliberately varying batching to develop imperfect mixing. Imperfect mixing could be produced in many ways which would be of no interest in the daily routine of concrete making.

If the batches were identical, some explanation should have been made of values for s/c in the "Mixer Dumped" test as high as 182 per cent for Test C as compared with a high of 105 per cent in Test A.

Table 6. Chemical and Mortar Tests on 36 Samples. Important data omitted; original mortar test weights a, b, c; mixing time; method of sampling; basis of chemical and mortar analyses.

The method used for computing mortar tests has been shown to be erroneous.

Mortar tests showed spreads of 0 to 8 per cent; average 3 per cent; the writer's analysis of tests in the original paper indicates that the average mean variation will be less than 1 per cent.

Chemical analyses show average spread of about 2½ per cent; this means that the mean variation in the three samples was about 1 per cent. This should be compared with 1 per cent for the mortar tests.

When properly interpreted each method gives a true measure of mixer performance and both show a similar high degree of uniformity in the fine materials in the batches.

We conclude that the concrete was thoroughly mixed long before the samples were taken and that the small variations are due solely to errors of sampling and testing.

Table 7. Concrete Containing Silicious Admixture. Important data omitted: original mortar test weights a, b, c; method of sampling.

The six highest s/c values are given in the first column below, followed by the corresponding variation in the chemical analysis:

Max. Variation		Ratio
s/c	Chemical	
7	3	2
7	3	2
7	1	7
10	2	5
22	2	11
37	3	12

Six out of 31 or 19 per cent of the samples gave absurd values for s/c as compared with the more dependable chemical analyses. Values for s/c from the mortar test 5 to 12 times that given by chemical analysis, do not show a satisfactory concordance between the two methods and throws serious doubt on the value of the mortar test. It serves no useful purpose to average the 31 mortar tests, when such wide discrepancies exist in individual values.

These results are similar to those in Table 6. The maximum variation in three samples by chemical analysis ranged from 0 to 5 per cent; average 2 per cent; this no doubt means total spread between low and high values; the average mean variation will be about 1 per cent, since chemical analyses are not affected by "corrections" in sand 0-100.

The chemical analysis shows that all batches were thoroughly mixed long before the samples were taken; the mortar tests confirm this, except for erratic tests. The small variations found were no doubt due entirely to unavoidable errors in sampling and testing.

Table 8. Hypothetical Samples. Gives no experimental data, hence it is of no interest in this discussion.

Table 9. De-Graded Batching Tests. Important data omitted: original mortar test weights a, b, c; method of sampling.

Test 1 was thoroughly mixed at some time between 1 and 1½ min. This is the control batch; it supports the writer's previous conclusion that regular Grand Coulee concrete was over-mixed at 2 min.

Test 2 shows better mixing at 1½ than at either 2 or 2½ min., and that the batch was sufficiently mixed at 1½ min.

Test 3 shows better mixing at 1 than at either 1½ or 3 min., and that the batch was sufficiently mixed at 1 min.

Test 4 shows concrete thoroughly mixed between 2 and 2½ min.

This table shows mortar test to give contradictory results and to be extremely undependable.

Mr. Patch refers to this table as authority for his statement (p. 180-17):

> It was frequently found in the large mixers when first tested that sand and cement proportions were satisfactory, but water distribution varied tremendously. The complete method reveals this condition sufficiently well. (See Table 9.)

Only one out of four tests in this Table shows a w/c higher than s/c in the first test. The table flatly contradicts the statement of Mr. Patch.

It will be noted that in spite of Mr. Patch's specific reference to the "complete method," the complete method was not used, since samples were taken only from front and back of the mixer.

Table 10. Mixer Efficiency Results. Important data omitted: original mortar test weights a, b, c; method of sampling; time of mixing.

Tests computed on a basis which has been shown to be erroneous.

The table gives the results of 84 tests carried out in 7 days; four tests must be excluded as not typical, giving abnormally high values due to errors in batching or other unusual conditions. The remaining 80 tests gave spreads in w/c or s/c ranging from 0 to 10 per cent; average spread 3.5 per cent. This indicates that the concrete was mixed to a mean variation of less than 1 per cent; or a greater refinement than is warranted. The small variations found can be fully accounted for by unavoidable errors in sampling and testing.

Running down this table one is struck by the violent fluctuations in the errors of a given mixer; the first s/c column shows the following errors in succession: 1, 4, 5, 1, 0, 6, 1, 8, 5, 2. The first and each third value thereafter are from the night shift. It is difficult to believe that these rythmic variations actually occurred in the mixer, hence we must inquire elsewhere for the answer.

Below are the s/c errors as indicated by the 80 tests:

Shift	s/c Error of Mixers				
	N.	E.	S.	W.	Av.
Night	1.9	2.2	3.1	2.4	2.4
Day	3.6	4.1	3.7	2.6	3.5
Swing	5.1	3.7	5.0	4.3	4.5
Av.	3.5	3.3	3.9	3.1	3.5

The averages at the right show the errors of the mixers classified with reference to the 3 shifts; the bottom line shows averages for the 4 mixers.

It is clearly shown that the mixers were *considerably more efficient at night than during the day.* We suspect that this peculiar result was due to the "personal equation" of the tester.

The comparison as between mixers and testers for 1 week is:

Spread of 4 mixers	26 per cent
Spread of 3 testers	87 per cent
Mean variation of 4 mixers	7 per cent
Mean variation of 3 testers	20 per cent

Much of the error charged to mixers is in the "personal equation" of the tester; errors of testers are three times as great as those of mixers. This suggests the need of a test for testers, rather than for mixers.

Following is the distribution of the tests with reference to w/c and s/c:

35 times w/c was high	44% of total
31 times s/c was high	38% of total
14 times w/c and s/c same	18% of total

The above contradicts the implication of the statement by Mr. Patch (p. 180-17) quoted above under Table 9.

The writer was not concerned with mixers when "first tested," but only with the routine tests given by Mr. Patch and in the "Manual;" the latter tests are dated Sept., 1936. Naturally if 10,000 tests are made, many things may occur "frequently." Table 10 shows that in only 44 per cent of the cases was the w/c the determining factor; similar results are shown in other tables.

The above tests show that there is no basis for criticising the writer's computation of mixer performance by a method which disregards the distribution of water and considers only cement and sand.

Fig. 1. Effect of Time of Mixing on Strength and Slump. The diagrams show the effect of time of mixing on strength and slump; however they seem to have no bearing on the questions at issue, since no mixer tests are given; apparently Mr. Patch now wishes to include concrete strength and slump as measures of concrete mixing; no such criteria are found in his original statement of assumptions.

Let's look at Batch 1, using 6-in. aggregate; the only one which has any bearing on Grand Coulee practice:

Time of Mixing	Strength	Slump
2	88	4½
20	100	1¼

Mr. Patch concludes that this concrete was improved by long mixing. Information published in the *Engineering News-Record* Oct. 15, 1936, shows that at Grand Coulee increasing the concrete slump 1 in. required the addition of 40 lb. of water to a 4-yd. batch. There is 1505 lb. of cement in a batch, hence each inch the slump is lowered will enable us to lower the w/c by

$$\frac{40}{1505} = .027$$

The w/c in the original paper was .564. Suppose we lower the slump 3¼ in. (from 4½ to 1¼); this will enable us to reduce the w/c of the batch by 3.25 x .027 = .09; this is a net decrease in the w/c of 14 per cent (from say, 0.65). The writer's figures indicate that this change would be accompanied by an increase in strength of about 14 per cent. The 88 per cent strength factor shown on the diagram for 2 min. mixing would then be increased by 14 per cent and would become 100 per cent. In other words, 2 min. mixing with the w/c required for a 1¼-in. slump would have produced the same *strength* and *slump* as that actually found after 20 min. mixing using the w/c necessary to produce a 1¼ in. slump at that time.

The writer fails to find in this test any evidence of "improvements beyond the 2-min. mixing period in both compressive strength and slump," claimed by Mr. Patch. In general concrete strength is increased by longer mixing, however Coulee aggregate presents an exceptional case in that the added mixing water necessary on account of excessive grinding neutralizes any advantage from longer mixing.

There are many pit-falls in the interpretation of tests made on wet-screened concrete.

METHOD OF SAMPLING FOR MIXER TESTS

It seems necessary to return to this subject which was considered briefly in my first discussion.

In the original paper Mr. Patch stated:

> Originally the mixers were stopped at the end of the prescribed mixing period, and three small samples were selected from front, center and back of the mixer respectively.
>
> The present procedure is to secure quickly three successive samples during the short period of mixer discharge.

The first method I referred to as giving "grab" samples. The essential features of grab samples are (1) that they be relatively small, (2) that they be taken without any blending with adjacent samples. The new method required the construction of a heavy movable arm attached to a support of the mixer building which is used to sweep a

container through the stream as the mixer discharges. These are not grab samples, in any sense of the word, but represent a ribbon of considerable length, hence avoid my criticism of the grab samples used in early tests. This change in sampling method, in my opinion, accounts in a large measure for the improved mixer efficiency found in later tests.

Mr. Patch still clings to the fiction that the samples secured by the new method come from front, center and back of the mixer, although it is impossible to tell where the samples originate, since the mixer is turning all the time.

Mr. Patch offers nothing to clear up points raised in my first discussion concerning sampling; in fact most of the new tables omit this important feature, without which it is impossible to interpret the mixer tests.

0-100 SAND BEFORE AND AFTER MIXING

The discussions of Messrs. Patch and Ruettgers reveal the utmost confusion of thought as to the origin, purpose and limitations of the "correction" factor used in attempting to account for the quantity of 0-100 sand in order to compute w/c and s/c of the mortar samples.

It should be pointed out in the beginning that the No. 100 sieve is not a precision instrument; individual openings may vary as much as 40 per cent from the average and still pass the standard specifications.

There are four cases to be considered in discussing the role of the 0-100 sand in concrete:

Case 1. Before-mixing quantity.
Case 2. Before-mixing uniformity.
Case 3. After-mixing quantity.
Case 4. After-mixing uniformity.

The BR "correction" hinges on the assumption that the after-mixing quantity of the 0-100 sand is immaterial, but that it is uniform in the three samples to four significant figures, hence Case 4, is the only one that applies. However, Messrs. Patch and Ruettgers now go back to the purely irrelevant Case 2, and for the first time inject this case into the discussion and base their arguments on it.

It will be necessary to discuss each of the above cases separately in order to remove this confusion.

Case 1. Before-Mixing Quantity of 0-100 Sand. Table 1 (Patch) shows the 0-100 sand at Grand Coulee to vary from 4 to 7 per cent in a few hours, with an average of 5.7 per cent for July, 1937. In the mixer test, no use was made of this information, nor is it possible

to make any use of it, since it is only the after-mixing quantity (Case 3) or uniformity (Case 4) with which we are concerned.

It has long been known that there is considerable grinding of aggregates in large batches; hence the before-mixing quantity of 0-100 sand cannot possibly be of any use, even if it were accurately known.

Case 2. Before-Mixing Uniformity of 0-100 Sand. Mr. Patch states (p. 180-14):

> Mr. Abrams says that data, a, b, and c are all that can be secured from the test. If this were true, the test would be of little value. The statement of Mr. Abrams presupposes that nothing is known about the fines in the sand, or whether they are mixed uniformly throughout the batch.

At the end of the above quotation Mr. Patch confuses Cases 1, 2 and 4 in the same sentence.

Table 1 gives information on both Cases 1 and 2 and shows sand samples on July 6 to 8 to have the uniformity in the 0-100 size as indicated by the following figures: 5, 4, 6, 6, 5, 6, 7 per cent. We find 50 per cent spread in adjacent samples; 4 and 7 per cent were found within a few hours, a spread of 75 per cent. If we consider that 4 includes 3.6 per cent and 7 includes 7.4 per cent, this spread becomes over 100 per cent within a few hours. We may readily conclude that the 0-100 sand as it goes into the mixer may show a spread of 100 per cent in the same batch. Mr. Patch neglects to call attention to the unusual conditions at Grand Coulee whereby the sand is separated into 3 sizes by means of classifiers, then recombined to a fixed grading.

Even if the original sand were perfectly uniform, this arrangement would be violently disturbed by the sudden injection of 120 gallons of water into a 4-yd. mixer. Whether or not the sand is uniformly graded before entering the mixer has little or no bearing on the after-mixing uniformity of the 0-100 sand.

The evidence of Table 1 (which Mr. Patch states "is typical") is in direct contradiction to the following statements:

By Mr. Patch (p. 180-9)

> The fact is that hundreds of sand tests taken monthly at the aforesaid mixing plants, it is very rare to find a change of more than 1 per cent in the −100 even in an entire day's run. The plant often runs for several days with less than 1 per cent variation in the −100 samples.

P. 180-12

> It has been indicated that Mr. Abrams' difficulty comes from analyzing the data presented on the false basis that the −100 material in the sand cannot be kept within 15 to 20 per cent variation.

P. 180-13

> . . . the various sizes of sand are well mixed before entering the mixer. . .

P. 180-16

. . . before the sand goes into the mixers at Coulee Dam, a variation of over 1 per cent in the —100 is very rare.

P. 180-20

Thousands of tests at Coulee Dam prove this statement false.

(That is, shows the spread in 0-100 sand to be less than 1 per cent).

By Mr. Ruettgers p. 180-21:

. . . field tests on several projects indicate that Mr. Abrams is in error in assuming that there is likely to be a large amount of relative variation in the minus 100 sand.

The *actual* range in 0-100 sand may be 5 to 7 times as wide as the limits set by Mr. Patch.

Incidentally, a, b, and c *are* the only data secured in the mixer test. The writer has shown that a satisfactory measure of mixer performance can be computed from b and c.

Case 3. After-Mixing Quantity of 0-100 Sand. Mr. Patch stated in the January JOURNAL with reference to the after-mixing fines in the sand: "Use 5 per cent if per cent of -100 sand is not known."

He actually used 8 per cent (although the sand was known to contain a much higher percentage) and computed the effects of this "correction" to four significant figures. This includes two entirely independent assumptions: (a) Case 3. Quantity of 0-100 sand, 8 per cent; (b) Case 4. Uniform at 100 sieve to four significant figures.

It appears from the statement of the "Manual" that Case 1, before-mixing quantity, has also been erroneously confused with the others. (See footnote p. 180-4, June JOURNAL).

If a purely arbitrary quantity (8 per cent) and perfection in uniformity (to four figures) are assumed, why make any pretense of using a "known" quantity of 0-100 sand?

If Mr. Patch had assumed different percentages of 0-100 sand, he might have computed a series of w/c and s/c from the example in the January JOURNAL as follows:

Assumed 0-100 Sand—%	Computed	
	w/c	s/c
0	.410	1.89
5	.459	2.22
8	.474	2.44
10	.497	2.63
15	.577	3.26
20	.695	4.30

Here is a method for approximately determining the after-mixing quantity of 0-100 sand, by moving along the table until the computed

w/c corresponds to the known w/c. The w/c of the batch was .564; hence a 0-100 sand of about 15 per cent is indicated after 2 minute mixing, and a new s/c of about 3.1 instead of the original 2.5.

It should be observed that the above computation did not require any assumption as to the uniformity of 0-100 sand, since we used only one sample of average weights a, b and c. We are merely computing two curves by starting with certain integral values of 0-100 sand and determining the corresponding w/c and s/c.

It is odd that the BR paid no attention to the after-mixing *quantity* but now gives so much attention to *before-mixing uniformity* of the 0-100 sand. Apparently Mr. Patch prefers to ignore the fact that a "sand-cement" was produced by excessive mixing. This subject is discussed more fully below.

Case 4. After-Mixing Uniformity of Sand 0-100. The before-mixing uniformity of 0-100 sand (we have seen that this probably showed a spread of 100 per cent in the same batch at Grand Coulee) gives no assurance whatever of after-mixing uniformity, due to: (1) Violent disturbance from entry and spreading of mixing water; (2) non-uniform mixing; (3) non-uniform grinding in mixer; (4) sand clinging to cement.

The after-mixing uniformity of the 0-100 sand was not determined; it cannot be determined any closer than the accuracy of a No. 100 sieve (limit about 1 per cent of the total sand). If 8 per cent is assumed, this includes the range 7½ to 8½. My first discussion showed that all BR mixer efficiencies are subject to a correction of 4 per cent in either direction on account of this unavoidable spread in the sieve test.

It was pointed out above that in assuming 8 per cent 0-100 sand after mixing, Mr. Patch also assumed that this material was uniformly distributed in the three samples to an accuracy expressed by four figures. My discussion under "A Case of Significant Figures" gave further details of this fallacy.

It should be reiterated that my criticism of the mixer test was not based on whether the after-mixing *quantity* of 0-100 sand was correctly assumed (and it never is), but on the erroneous assumption that it is *uniform to four figures.*

"SAND-CEMENT" IN GRAND COULEE DAM

Probably more harm was done by long mixing at Grand Coulee than would have resulted from considerable under mixing. A 4-yd. batch contains about six tons of coarse sand and gravel up to 6 in.; this material with water makes an ideal grinding machine.

The original sand contained about 6 per cent 0-100; after mixing 2 minutes there was about 15 per cent; further mixing produced a "sand-cement," as indicated below:

Time of Mixing Min.	Per Cent of 0-100 in Sand	Estimated Make-up of "Sand-Cement"—%	
		Sand	Portland
0	6	—	—
2	15	27	73
5	25*	38	62
10	35*	47	53
15	40*	50	50

*Estimated from amount ground during the first 2 min. mixing.

A s/c of 2.5 for the original batch was used. The estimated values for grinding are believed to be conservative; the true values may be considerably higher.

The effect of this grinding in reducing slump of Coulee concrete is shown in Fig. 1, particularly in the first chart using 6-in. aggregate.

During early concreting at Grand Coulee mixing up to 5 or 10 min. (or more) was required (Patch); hence the lower portion of the Dam is made of a "sand-cement" consisting of intimately ground fine sand and portland cement in proportions up to about 50-50 per cent. The resulting cement is similar to the sand-cement used 25 years ago in Elephant Butte and Arrowrock Dams.

This is an expensive method of producing a "sand-cement." There is some grinding of the cement, however if a finer cement was desired or if a sand-cement was wanted, this work should have been done in a cement mill and not in the concrete mixers.

GRAND COULEE CONCRETE

Data on Grand Coulee concrete may be tabulated as below:

	Spread—%	Mean Variation—%
Within a Batch (sand-cement ratio)		
Specification limit (Shasta Dam)	10	—
Max. for Sept., 1937 (Jan. Jl.)	15	—
Av. " " "	3.8	1**
Max. 36 tests (Table 6)	8	—
Av. " " "	3	1**
Av. chemical tests (Table 6)	2½	1
Max. Nov. 4-10, 1938 (Table 10)	8*	—
Av. " " "	3	1**
Batch to Batch (concrete strength)		
28-day July, 1937, single cylinders	50	—
" " " Av. twin cylinders	43	—
" " " 3 successive samples	—	11
8-hr., Sept., 1937 (Jan. Jl.)	61	—
28-day Sept., 1937 (Jan. Jl.)	70	—
Strength of cement (Patch)	40% or more	—

*Two values of 11 and 15% not included, since they were attributed to errors in batching or other cause which must have been recognized in some other manner.
**Estimated from tests by Patch and in Manual.

Spread. The BR regularly accepts spreads in cement and concrete strengths (batch to batch) which are 10 to 17 times those demanded or regularly secured by the concrete mixers (within a batch).

Mean Variation. The BR accepts mean variations in concrete strengths (batch to batch) which are 11 times those regularly secured within the batch.

The best efforts in the testing of concrete cylinders do not remotely compare in uniformity with the regular performance of the concrete mixers.

It is difficult to understand the claim of Mr. Patch that

> . . . cement from different mills, and even from different runs of clinker from the same mill, differ 40 per cent or more in strength.

Cement from the various mills goes through an elaborate blending plant before it is sent to the mixers. If variations in cement are responsible for the enormous spread in concrete strengths, the problem is in the cement mills or blending plant and not in the concrete mixers.

COULEE CONCRETE OVER-MIXED

The writer's first discussion stated with reference to Grand Coulee concrete; (p. 180-6):

> The evidence available indicates that the concrete was probably grossly over mixed when the samples were taken after 2 min. mixing.
>
> There is nothing in the report to show that the concrete was not sufficiently mixed at 1 min.

Messrs. Patch and Ruettger both assert that no evidence was presented to show this. They overlooked my heading "What Can be Learned from Grand Coulee Test Data?" where it was shown that the average mean variation of the three samples each from two BR batches was 1 per cent. When the fine material in a mixer is uniform to this degree, yet the strength in different batches spread 70 per cent in a month, the writer concludes that the concrete was sufficiently mixed at one minute.

BR writers claim that I offered no evidence of over-mixing of concrete at Grand Coulee; it is no doubt just a strange coincidence that mixing times were omitted from most of the new tables presented by Mr. Patch.

As discussed above, the chemical analysis in Table 6 and 7 offer additional proof that the concrete was mixed to a mean variation of 1 per cent or less. Mixing time given for Table 7 only, (2½ min.). The writer concludes that the concrete was over-mixed when the samples were taken.

The "control" test in Table 9 shows that the 4-yd. batch was thoroughly mixed before the samples were taken at 1½ min.

A week's run of mixer tests in Table 10 shows that the batches were mixed to a mean variation which averaged less than 1 per cent. Time of mixing not given, presumably 2 min.; whatever the mixing time, the concrete was over-mixed when the samples were taken.

A mean variation of 11 per cent in concrete strength (batch to batch) as compared with 1 per cent in the batches shows that too much emphasis has been put on mixing.

If concrete is mixed to a variation of 1 per cent in 2 min., the writer considers it *extremely likely*, in the light of the data available, that it was sufficiently mixed at 1 min. The 1 per cent mean variation found so frequently in the reported mixer tests is barely sufficient to cover unavoidable errors of sampling and testing.

We may ask a question somewhat similar to one in my first discussion: "If concrete strength shows a mean variation of 11 per cent (as determined by 3 successive samples), in spite of unprecedented control, why is it necessary for a given batch to be mixed to a mean variation of 1 per cent (as determined by three mortar samples)?'

SIX NEW CRITERIA OF MIXER EFFICIENCY

I quote (for the second time) the basis of the mixer test as stated by Mr. Patch in the January, 1939, JOURNAL:

> The mortar-mix test now in use was based on the assumption that if the sand, cement and water were uniformly mixed throughout a concrete batch, and there was no noticeable segregation of coarse aggregate, the mixing was satisfactory.

The following requirements are now introduced in an effort to justify the mixer test or the mixing times required:

By Mr. Patch—

(1) Concrete strength;
(2) Concrete slump;
(3) Sand well-mixed before entering the mixer.

By Mr. Ruettgers—

(4) Workability at point of placing;
(5) Position of consistency-meter pen;
(6) BR practice corresponds to that of others.

Nothing was said in the original assumption concerning strength, slump, well-mixed sand, the position of the consistency-meter pen, or the practice of others.

When this method was promulgated for general use in the BR Manual after four year's experience, nothing was said about strength, slump, well-mixed sand, or the position of the consistency-meter pen.

When the method was written into BR specifications for large dams, nothing was said about strength, slump, well-mixed sand or the position of the consistency-meter pen.

Messrs. Patch and Ruettgers now wish to add six new criteria of mixer performance to the one originally announced.

A CONSTRUCTIVE CRITICISM

Mr. Ruettgers states that my criticism was "of a destructive character." On the contrary, I indicated how the mixer test data could be used to give a true measure of mixer performance.

The following is quoted from Mr. Ruettger's discussion (p. 180-21):

> He then proceeds to explain how a *small variation* between samples in the percentage of minus 100 sand may greatly affect the computed spread in sand-cement ratio, and, finally, he presents a method of stating results which he states "gives an ideal measure of mixer efficiency." However, field tests on several projects indicate that Mr. Abrams is in error in assuming that there is likely to be a *large amount of relative variation* in the minus 100 sand.

In order to make his point Mr. Ruettgers finds it necessary to shift from "small variation" in one sentence to "a large amount of relative variation" in the next. There should have been no question as to the variation considered in my discussion. My interpretation of the mixer test began with:

> The paper is quite confusing as to what was meant by 8 per cent sand 0-100. From the round numbers used, we would be compelled to conclude that 8 per cent includes the range 7½ to 8½ per cent.

And continued with the question (p. 180-4):

> What would be the effect on computed mixer efficiencies if we considered that 8 per cent means anything from 7½ to 8½ per cent?

Mr. Ruettgers might readily have concluded that a 1 per cent range was being considered; however he did not choose to throw any light on the above questions.

The writer's first discussion presented a method of computing mixer efficiency which could be derived by simple calculations from the original b and c, where b is immersed weight of mortar sample, and c is the immersed weight of sand (after washing fine materials through a No. 100 sieve).

The method consists of subtracting c from b and dividing by b. Two examples were worked out from the data available; the average mean variation in the distribution of 0-100 solids was 1 per cent.

My method was based on the assumption that if the *total amount* of material through the 100 sieve (cement, sand and silt) was uniform after mixing, the distribution of this material as between cement and fine sand would be uniform also, since the cement constitutes about

80 per cent of this size and offers the greatest resistance to mixing. After-mixing variations in the 0-100 sand would affect only about 20 per cent of the total, hence could be disregarded.

This method does not require the "correction" which has proven disasterous in the BR mixer test.

The routine mixer tests carried out at Grand Coulee during the past 4 years have accomplished little more than to determine 24 times daily the errors inherent in the test method.

A NOVEL METHOD OF MEETING AN ENGINEERING CRITICISM

My criticisms of the BR mixer test were supported by specific figures based on two tests; the only data available to me. The BR writers devoted 17 pages to my discussion; it is notable that neither chose to give a few lines to the important feature of my criticisms. Mr. Patch disposed of my heading "Computations Based on a Fallacy," which closed with "When 8 per cent 0-100 sand was assumed for all samples (correct to four places) this assumed also that the batch was already uniformly mixed," by stating "that the various sizes of sand are well mixed before entering the mixer but the cement is batched independently." This of course ignores the whole point of my criticism. I have shown above that the before-mixing grading of sand has no bearing on the problem, and in fact had not been mentioned before.

Mr. Ruettgers devotes 15 lines (out of 5 pages) to the real point of my criticism; eight of these are given over to a recital of the various steps in my analysis of the fallacy of the BR mixer test; he then disposes of the entire matter in the following two sentences, p. 180-22:

> However, field tests on several projects indicate that Mr. Abrams is in error in assuming that there is likely to be a large amount of relative variation in the minus 100 sand. In any event, his "ideal measure" would likewise be affected by such variation, and, moreover, it does not measure separately the variations in the sand-cement and water-cement ratios.

The first sentence is non-commital in the extreme; the second seems to say that while there is no variation in the 0-100 sand, my method is unsound because it would nevertheless be affected by such variation.

What is a "large amount of relative variation"?

Was the minus 100 sand tested before or after mixing?

Why did he not give a few examples to show how my ideal measure was affected?

Were the indications of the field tests on several projects similar to those reported from Grand Coulee Dam?

Messrs. Patch and Ruettgers presented in the June JOURNAL hundreds of new values in defense of the mixer test, yet it is notable

that *not one original weight was given*, from which an independent computation of mixer performance could be made.

The original mortar test data have been "corrected" in computing weights of water, cement and sand, which are known to be wrong; these values are then concealed by working out water- and sand-cement ratios; and further concealed by computing mixer efficiency factors. In other words all mixer tests are reported in a "code" which defies translation.

Most of the writer's deductions on the BR mixer test were based on the single mixer test which was given in the January JOURNAL; some use was made of the test in the "Manual." Mr. Patch does not seem inclined to release another set of original weights.

Mr. Patch states (p. 180-8):

> Abundant physical and chemical data are on file to convince anyone who may be interested as to the facts.

Instead of giving the original data, Mr. Patch finds it expedient to invite JOURNAL readers to visit Grand Coulee Dam.

My first discussion contained comments on BR sampling methods and time of mixing. Is it accidental that these data are missing from most of the new tables?

WHAT ARE THE SIX ARITHMETICAL ERRORS?

Instead of presenting engineering data Mr. Ruettgers attempts to discredit the writer's method of computing mixer performance by the following (p. 180-22):

> Incidentally, there appear to be six arithmetical errors in the ten figures which he presents in illustrating his index.

Mr. Ruettgers should have been able to determine *definitely* whether or not there were errors; only subtraction and division were involved, using BR data which he had before him.

Why did he not devote a line or two to pointing out what the errors were and what, if any, effect they had on my conclusions?

Why did he not apply my method to a few of the thousands of tests at his disposal and give the results?

CONTRADICTIONS

The writer has already called attention to a number of conflicting statements and instances in which the statements made are contradicted by the data presented. It is impossible to refer to them all. The following three conflicting statements are found on one page (p. 180-18):

> . . . 10 per cent variation requirement is not extreme or impractical. . . .
>
> As a matter of fact the mortar test spread is not held to 10 per cent variation . .

. . . tests were considered satisfactory unless the variation in the 3 samples was over 10 per cent . . .

Mr. Patch seems to take both sides of the argument at the same time with reference to the 10 per cent spread as a criterion for mixer efficiency, in spite of the fact that this value has been written into BR specifications for three of the largest concrete dams in the U. S.

A careful study of the discussions of Messrs. Patch and Ruettgers shows that practically every statement made in defense of the BR mixer test is contradicted by the tests presented by them.

MIXER TESTS ON DAMS

Mr. Ruettgers states (p. 180-20):

. . . the test was subsequently tried out with marked success on the mixers used in the construction of four other major Bureau dams.

He then gives certain details for four dams; only two of these will be referred to.

Dam B. "Replaced worn mixer blades."

It does not seem that an elaborate test method is necessary to determine when worn blades should be replaced. It has been known for more than half a century that mixer blades wear out. A farmer plowing in the field knows without an elaborate staff when his plow point has worn out; having put on a new one, he does not make tests three times daily for the next six months in order to determine when a new point is needed.

Dam D. "Releveled mixer."

Elaborate tests would not seem necessary; a hand level which Woolworth's sell for 20¢ would show whether the mixer was level.

No doubt the above dams were among the jobs which Mr. Patch had in mind when he stated (p. 180-8):

. . . the test has been used successfully to effect *almost phenomenal* correction of deficiencies in concrete mixing on a number of other Bureau projects."

MIXING TIME ON LARGE DAMS

Mr. Ruettgers tabulates the mixing times on 28 large concrete dams built from 1913 to date, to show ".....that Bureau practice in this respect corresponds with that of others." No information is given on cement factors, consistency, size or type of aggregates; these must be taken into account in comparing mixing on other dams with that at Grand Coulee.

If the BR wished to depend on precedent, why did they spend a hundred thousand dollars (as at Grand Coulee) in testing mixers? The older dam builders had only practical experience to guide them.

The BR should now have a better basis for defining mixing requirements than precedent established on dams built as long as 26 years ago.

The writer's first discussion stated that there was no rational basis for the mixing times required by BR specifications; the figures given by Mr. Ruettgers furnish an excellent corroboration. 1½ min. was required for the 4-yd. mixers at Owyhee, while at Boulder and Grand Coulee, mixers of the same capacity using similar mixes and aggregates were required by the specifications to run for 2½ min.

MIXING SPECIFICATIONS FOR SHASTA DAM

The specifications for Shasta Dam (BR No. 780) now under construction in California were published about May, 1938. The following is from Section 116:

> . . . the Government reserves the right to increase the mixing time when, in the opinion of the contracting officer, the charging and mixing operations fail to result in the required uniformity of composition and consistency within the batch and from batch to batch or when test samples of concrete taken from three locations such as the front, center, and back of the mixer show a difference of more than 10 per cent in the sand-cement or water-cement ratios. . . .

The BR has recommended their mixer test for general use as indicated by its inclusion in their "Concrete Manual" of 1938, Revised Edition 1939, and in the JOURNAL.

The above clause* may be expected to reflect the official attitude of the BR with reference to the mixer test, after having used it at Grand Coulee for two or three years and having prepared the "Manual" which devoted four pages to the test.

In the Shasta specification the mixer test ranks a little below "the opinion of the contracting officer."

CONCLUSIONS

A further study of the Bureau of Reclamation concrete mixer test as used at Grand Coulee Dam leads the writer to the following conclusions:

(1) Useful mixer data have been misinterpreted as a result of an error in elementary arithmetic.

(2) The mixers have been penalized for small variations due to unavoidable errors in sampling and testing.

*A similar clause is found in the specifications for Friant Dam (BR No. 863) July 1939.

(3) The concrete was over-mixed due to an erroneous criterion for mixing.

(4) Mixer performance has been held to an unreasonable refinement of the order of 1 per cent; whereas three successive samples show mean variations in concrete strength of 11 per cent and different batches show a spread in concrete strength of 70 per cent in a single month.

(5) The tests as regularly carried out at Grand Coulee during the past four years have accomplished little more than to determine 12 or 24 times daily the errors inherent in the test method.

(6) In presenting the new data Mr. Patch has with-held pertinent information, particularly original mortar test weights, methods of collecting mortar samples and time of mixing.

(7) Much of the error charged to mixers is in the "personal equation" of the tester; errors of testers are 3 times as great as those of the mixers. This suggests the need of a test for testers, rather than for mixers.

(8) The principal arguments of Messrs. Patch and Ruettgers in defense of the BR mixer test are based on the before-mixing uniformity of the sand, which has little or no bearing on the after-mixing uniformity.

(9) Messrs. Patch and Ruettgers now wish to add six new criteria of mixer performance to the one originally announced.

(10) Practically every statement made by Messrs. Patch and Ruettgers in defense of the BR mixer test is contradicted by the tests submitted.

* * *

The misinterpretation of the mixer tests might be dismissed by engineers outside the BR as a curious error, were it not for the knowledge that:

(a) The application of this test at Grand Coulee has already caused the Government and the contractors to spend hundreds of thousands of dollars needlessly.

(b) This erroneous method has been promulgated for general use by publication in the BR "Manual" which is offered for sale to the public.

(c) A sand-cement was unintentionally used in the lower portion of Grand Coulee Dam due to over-mixing as a result of a misinterpretation of mixer data.

AUTHOR'S CLOSURE*

I received the 29-page discussion by Mr. Abrams, July 24 and the subsequent voluminous corrections and revisions, August 7. Obviously any comprehensive reply by August 11 would be impractical, even if I had no other responsibilities. I gave Mr. Abrams the benefit of assumed sincerety in the first discussion, and went to considerable trouble to clarify the purpose and soundness of the procedure, all of which data seem to have been misconstrued or ignored by Mr. Abrams, leaving the whole argument again pivoted on his own unsupported assumptions. I see no further value to anyone in continuing the discussion.

As far as the test, and also the discussion submitted by Mr. Abrams is concerned, I would simply suggest that anyone who has a practical interest in the matter just try the test. If it cannot stand on its own merits with reasonably intelligent application, it should be either improved or discarded. So far we know of no case where it has failed in trial.

*Received by the Institute Aug. 15. This letter by Mr. Patch under date of Aug. 7, constitutes the author's closure of discussion of his paper published in the Jan. 1939 JOURNAL and previously discussed in the June JOURNAL.

Vol. 35 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *JANUARY 1939*

Tests of the Potential Durability of Horizontal Construction Joints*

By Chas. E. Wuerpel†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

This paper is a contribution to the effort to make horizontal construction joints, particularly in hydraulic structures, which are not planes of weakness and deterioration. The author describes the satisfactory use of air and water jets on the surface of concrete lifts.

The potential resistance to frost action of horizontal construction joints in hydraulic concrete structures has been investigated by the Concrete Laboratory operated by the North Atlantic Division Office of the U. S. Engineer Department. Recognizing the limits to which conclusions may be drawn from studies of small specimens prepared in the laboratory, the investigation was conducted in the field on sections of concrete placed in connection with certain flood control structures in the North Atlantic Division of the U. S. Engineer Department.

Sixty-seven cores of 5-and 6½-in. diameter were extracted from the test sections in such manner as to include a horizontal construction joint in each specimen. Accurate elevation records were kept so that the position of the joint in each core would be established within ¼ in.

The cores were shipped to the laboratory at West Point, where they were divided into lots and subjected to initial freezing and thawing at ages varying from 14 to 90 days. The different ages were used to determine their effect upon the durability of the joints.

Only one of the sixty-seven cores developed a joint failure, although most of the cores showed evidence of general deterioration after 70 freezing and thawing cycles. Sone of the cores initially frozen at an

*Received by the Institute Oct. 31, 1938.
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age of 14 days disintegrated completely after less than 70 cycles, without showing signs of joint distress prior to failure of the core.

The characteristics of the concrete mixtures used in the investigation are shown as follows, on various works.

Basic Data, Construction Joint Studies

Area 1, Waterbury Dam—Wire brush method. No grout used. 14 cores extracted. Mix data: 1-2.28-5.32, w/c = 6.0, C.F. 4.86 sacks, slump 1 in. Compressive strength (cores) = 4390 p.s.i. at 28 days.

Area 2, Waterbury Dam—Air and water jet method. No grout used. 16 cores extracted. Mix data: 1-2.43-3.31, w/c = 6.4,C.F. = 6.0 sacks, slump 3 in. Compressive strength (cores) = 3580 p.s.i. at 28 days.

Area 3, Waterbury Dam—½ air and water and ½ wire brush. No grout used. 6 cores extracted. Mix data: 1-2.28-5.32, w/c = 6.0, C. F. = 4.86 sacks, slump 1 in. Compressive strength (cylinders) = 3900 p.s.i. at 28 days.

Area 1, Springfield—Air and water jet method. Layer of grout ½ inch thick used. 5 cores extracted. Mix data: 1-2.44-4.46, w/c = 6.5, C.F. = 5.23 sacks, slump 6 in. Compressive strength (cores) = 3220 p.s.i. at 28 days.

Area 2, Springfield—Air and water jet method. No grout used. 5 cores extracted. Mix data and compressive strength = Same as Area 1, Springfield.

Area 3, Springfield—Air and water jet method. Layer of grout ½ inch thick used. 7 cores extracted. Mix data: 1-2.44-4.46, w/c = 6.0, C.F. = 5.23 sacks, slump 2 in. Compressive strength (cores) = 4475 p.s.i. at 28 days.

Area 4, Springfield—Air and water jet method. No grout used. 7 cores extracted. Mix data and compressive strength = Same as Area 3, Springfield.

Area 5, Springfield (wall stem)—Air and water jet method. ½ of area no grout; ½ of area ½ in. of grout used. 7 cores extracted. Mix data: 1-2.44-4.46, w/c = 6.25, C.F. = 5.23 sacks, slump 4 in. Compressive strength (cylinders) = 4000 p.s.i. at 28 days.

The appearance of representative cores after 70 cycles of freezing and thawing is shown in Fig. 1. The joint planes in these cores are about ½ inch below the white string which holds the number plates.

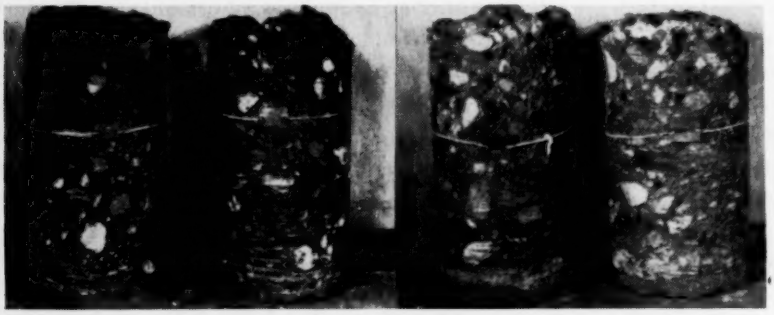

FIG. 1—REPRESENTATIVE 6½ INCH FIELD CORES AFTER 70 CYCLES OF FREEZING AND THAWING. EACH CORE CONTAINS A HORIZONTAL CONSTRUCTION JOINT

FIG. 2—CORE FROM HIGH-LIFT WALL SHOWING BELOW-JOINT DETERIORATION

Particular attention is invited to the photographic illustration of the core in Fig. 2. This specimen is representative of a group of cores extracted from a thin wall section where the height of the lift to the joint plane was approximately 10 feet. The narrow section and steel reinforcement necessitated the use of a moderately wet mixture, 4-in. slump, to permit consolidation of the concrete by hand-placing methods. The deterioration of the concrete below the construction joint is much greater than in the remaining portion of the core, but the joint, itself, remained sound. The cores removed from the low lift sections showed no evidence of this below-joint deterioration.

The homogeneity of concrete containing horizontal construction joints properly cleaned by the air and water jet method is shown in the reproduction of a full scale photograph of a section of a core on the left in Fig. 3 (here reduced to approximately 40 per cent). The position of the joint plane is shown by the small opposing arrows. The upright arrow in the lower left indicates the approach to the top of the lower lift of concrete. The specimen on the right of Fig. 3 is similar, except that the concrete was wet and the joint was improperly cleaned. The segregation and joint porosity are clearly indicated in the latter core.

Half of the test joint surfaces prepared at the Waterbury Dam were cleaned with the high pressure air and water jet shown in Fig. 4. The

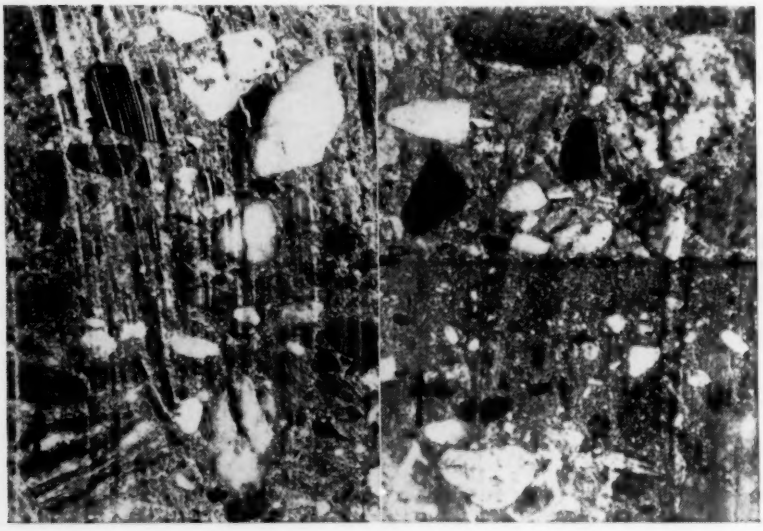

FIG. 3—FULL SCALE PHOTO OF JOINTS IN TWO SPECIMENS HERE REDUCED TO APPROXIMATELY 40 PER CENT. (LEFT) HOMOGENEOUS CONDITION OF A PROPERLY PREPARED JOINT. (RIGHT) IMPROPERLY PREPARED JOINT

other half of the joint surfaces were cleaned with wire brushes. All of the Springfield joint surfaces were cleaned with the high pressure air and water jet. Fig. 5 shows a joint surface being cleaned by this method. The fact that the durability of the wire-brushed joints equalled the air and water jet-cleaned joints in this investigation is simply a tribute to the thoroughness of the laborers who wielded the wire brushes—they became aware of the object of the investigation and were unusually zealous in their work.

The superiority of the air and water jet method to other methods of cleaning horizontal construction joints has been illustrated, with joint permeability and flexural strength as criteria, by the Engineering Materials Laboratory of the University of California.‡

The following conclusions are indicated from this limited investigation:

1. Joints which have been thoroughly cleaned are as resistant to frost action as the contiguous concrete.

2. Cutting the upper surface of lifts with a high pressure air and water jet, effectively removes the inert materials and air froth which inhibit bond.

‡JOURNAL, Amer. Concrete Inst., May-June 1934. *Proceedings* Vol. 30, p. 422.

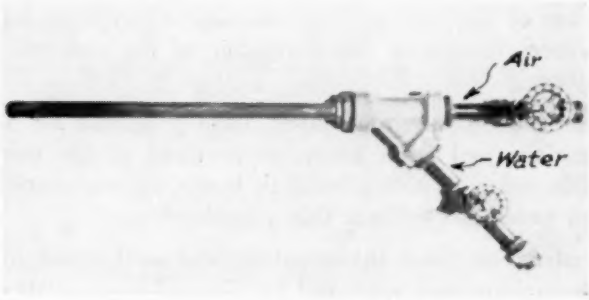

FIG. 4—HIGH PRESSURE AIR AND WATER JET

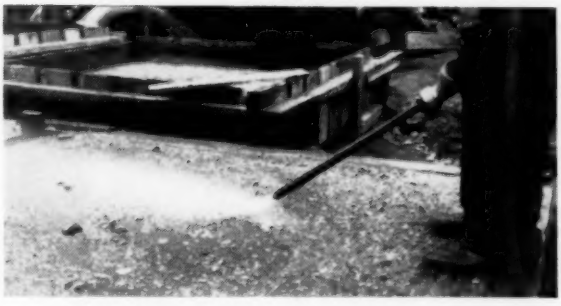

FIG. 5—CLEANING HORIZONTAL JOINT SURFACE WITH HIGH PRESSURE AIR AND WATER JET

3. Experience gained in this investigation indicates that the proper time for cutting with the jet is about the time of final set of the concrete. Temperature and humidity affect the time of set to such an extent, however, that it is impracticable to establish arbitrary times for optimum effectiveness of cutting. The proper time for jet application seems to be when the edge of a boot drawn across the concrete surface leaves a groove without dislodging coarse aggregate. Disturbance and removal of a moderate amount of coarse aggregate particles is preferable to ineffective cutting.

4. The air pressure used with the jet should not be less than 100 pounds. If lesser pressures are used there is a tendency to compensate for reduced efficiency by the use of an excessive amount of water, or, by applying the jet well before final set has been attained. Only enough water should be used to form a finely divided spray with sufficient impact to dislodge the film of inert and porous matter which rises in greater or lesser degree to the surface of all concrete.

5. The age of the joint had no noticeable effect upon its resistance to frost action, except as the durability of the concrete itself was affected by age.

6. While the ½ in. grout layer, usually spread on a hardened concrete surface just prior to commencement of the next lift, has incontestable value in aiding bond in joints, no superiority of joints so prepared were observable in this investigation.

The construction joint investigation was authorized by Lt. Col. C. H. Cunningham and approved by Colonel E. L. Daley, Division Engineer. The field work was made possible through the cooperation of Lt. Col. J. S. Bragdon and Major P. M. Ellman.

Discussion, to close in June 1939 JOURNAL, *should reach A. C. I. Secretary in triplicate by Apr. 1.*

JOURNAL
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Discussion of a paper by Chas. E. Wuerpel

Tests of the Potential Durability of Horizontal Construction Joints*

BY DUFF A. ABRAMS, RAYMOND E. DAVIS, ROBERT F. BLANKS AND BY THE AUTHOR

BY DUFF A. ABRAMS†

No feature of concrete dams deserves more attention at this time than the horizontal construction joints or the work-planes between successive lifts. Many of those in charge of dam design and construction appear to take the position that all the problems of joints have been solved, and treat these joints as if they were minor details which are automatically taken care of. In the writer's opinion these are fundamental errors.

The slip-shod methods now used in the design and construction of these joints is evidence of loose-thinking and of a willingness to sacrifice too much to the convenience of the construction forces.

Mr. Wuerpel's paper indicates a commendable interest in the subject; but the tests carried out at West Point have not cleared up this writer's grave doubts as to the merits of joints in which the unset concrete has been "cut" by air-water jets applied at high velocity.

Freezing and thawing tests on 14-day-old cores have little resemblance to the service conditions to which hydraulic structures are exposed. The important question is whether the joints are watertight. It is of little interest to know that the concrete will withstand 70 repetitions of freezing and thawing (or 700 repetitions), if a dam is to be floated downstream as soon as water has had time to find its way into the joints.

*JOURNAL, Amer. Concrete Inst., Jan., 1939; Proceedings Vol. 35, p. 181.
†Consulting Engineer, New York.

The language of Mr. Wuerpel's report is quite confusing with reference to the compression tests; he states that "67 cores .. .. .. were extracted in such a manner as to include a horizontal construction joint in each specimen," hence the writer concludes that the compression-test cores included joints. On this basis we have rearranged the strength data in the accompanying table:

Job	Cement Sacks	W/C Gal.	Slump in.	28-Day Strength p.s.i.
Cores, Wire-Brush				
Waterbury	4.86	6.0	1	4390
Cores, Air-Water				
Waterbury	6.4	6.0	3	3580
Springfield	5.23	6.5	6	3220
Springfield	5.23	6.0	2	4475
Av.	5.62	6.2	4	3760
Cylinders				
Waterbury	4.86	6.0	1	3900
Springfield	5.23	6.25	4	4000
Av.	5.05	6.1	2½	3950

The cylinder tests indicate that the concrete in the two jobs was of essentially the same strength at 28 days. The limited data indicate that the wire-brush cores are about 10 per cent stronger than normal and that the average of the air-water cores was about 5 per cent below normal. It will be noted also that the air-water cores were quite variable in strength.

Mr. Wuerpel's report would have been more instructive had he furnished information on:

(1) Type of cement, fine and coarse aggregates.

(2) Method of placing—vibration, etc.

(3) Height of lifts above joints sampled.

(4) Time between lifts.

(5) Deformations across the joints in compression tests.

(6) Strength of cores at different elevations above the joint.

(7) Water-tightness of joints, after exposure to high pressure for long periods.

(8) Compression strength of cores containing joints, after the freezing and thawing tests.

Reported tests on cores from other dams have shown a marked decrease in strength from the bottom to the top of a lift*; more information on this subject is needed.

It should not be assumed that joints are water-tight merely because they are not visible.

It may be worth while to trace some of the high-spots in the history of air-water cutting of unset concrete surfaces, particularly on large dams.

The method was invented by the contractors on Boulder Dam in 1933 as a labor-saving device. It was substituted by the U. S. Bureau of Reclamation for the specified method of cleaning the *hardened* concrete by ".. .. .. thorough scrubbing, brushing, chipping, sand-blasting, washing and blowing with compressed air." (Specification 519) and has been used on many of the largest dams built in the U. S. during the past 6 years.

The nonchalant manner in which this questionable method was adopted, specified and used is shown by the fact that it is mentioned in only the most casual fashion in the B-R publications prior to 1938. The July, 1936, edition of their "Manual for the Control of Concrete Construction" devoted one sentence to the subject (p. 201):

> The surface is prepared to receive the succeeding lift by the removal of all laitance and loose or defective surface concrete. This may be accomplished at the proper time during the setting period by means of jets of air and water applied at high velocity.

It was necessary to give the first sentence in order to bring out the meaning of "this" in the second sentence.

The method was specified for the foundation section of Grand Coulee Dam (No. 579), and was used until the completion of that contract early in 1938.

The writer made a study of this method during job tests carried out on a dam built under the supervision of the B-R in October, 1937, and concluded that the method:

(1) Was dangerous.

(2) Should never have been specified or used.

(3) Should be strictly prohibited.

The above conclusions and the information on which they were based have been in the hands of the contractors for the past 15 months. The air-water gun used was similar to that described by Mr. Wuerpel. The evidence from the job tests and observations was unmistakable; there have been indications that B-R engineers agreed with the writer's findings.

*See tests of cores from Ariel Dam, Trans. Am. Soc. Civil Engineers, v. 99, p. 913, (1934); also tests from Kanawha River work, *Civil Engineering*, Dec., 1938.

It may have been a mere coincidence, but it is interesting to record that the B-R immediately paved the way for quietly scuttling air-water cutting. This was done by introducing an alternate method in the specification (No. 757) for the second Grand Coulee contract, November, 1937. The method does not appear in the specification (No. 780) for Shasta Dam, published about May, 1938; in fact the wording is such as to prohibit the use of the air-water gun on unset concrete. More recently the method has been discontinued on other dams under construction by B-R.

The July, 1938 edition of the B-R "Manual" devoted several pages to a condemnation of the method and concludes:

> As a consequence of the considerations just discussed, there has recently developed a trend in engineering opinion in favor of omitting the initial cleanup and employing only a final cleanup treatment for horizontal concrete construction joints. Two recent Bureau specifications for large dams have stipulated this latter method.

"Initial cleanup" refers to air-water cutting.

The above sequence of events shows that the "trend" constituted a complete reversal in policy and that it was recent indeed. In other words, at the end of 5 years of active dam building the B-R learned that the method was dangerous, after having used it on some of the largest dams in the world, and devised the above quaint method of announcing their change in practice.

It is not clear why the B-R awaited a "trend in engineering opinion" on a vital feature of dam construction, when they had a million-dollar research department to determine the facts.

Engineers appear to know how to *design* concrete dams, but great difficulties arise when the attempt is made to *construct* a large dam with any assurance that it will behave according to the blue-prints.

A concrete dam is no better than its poorest construction joint. Better methods of designing and constructing these joints must be used, or radical changes must be made in design assumptions, if concrete dams are to be successful. It appears that recent "progress" has been in a backward direction. At this date (January, 1939) joint practice is still in a very unsatisfactory state.

A dam that is "nearly right" is not good enough. Public interest demands that concrete dams be built safely or not at all.

Concrete dam builders have not yet learned the proper uses of a number of the new tools which have recently come into their hands. The air-water gun is a handy tool on a large concrete job, but it should never have been used for cutting unset concrete at construction joints. A discussion here of the mis-uses of other tools would lead us too far afield.

Mr. Wuerpel has done well to bring this subject to the attention of the Institute. A full and frank discussion of construction joints is highly desirable. A more complete exposition of the writer's findings is in preparation for early publication. In the meantime it is hoped that these notes will serve as a STOP—LOOK signal to other dam builders who may have adopted air-water cutting without adequate investigation.

BY RAYMOND E. DAVIS*

Mr. Wuerpel's paper shows two things: first, that if the surface of an old lift of concrete is properly prepared, the construction joint is as resistant to freezing and thawing as are the adjacent lifts, and second, that when there is water gain the concrete in the top of a lift exhibits a decidedly lessened resistance to frost action.

The early investigations made at the University of California, referred to by Mr. Wuerpel, were made to determine the relative bond strength of construction joints where the old surface was, (1) untreated, (2) chipped and wire brushed, and (3) cleaned with the air water jet. The results of the investigation showed that the air water jet method of clean up, properly applied at some time before final set, was vastly superior to chipping and brooming of hardened surfaces. This led to the discontinuance of chipping at Boulder Dam, and to the general use of the air water jet.

In practice the air water jet method has been found to have two definite disadvantages:

1. To obtain a good bonding surface, the air water jet must be applied at just the right time; if applied too soon, the result is "over cutting" with many partially embedded pieces of coarse aggregate, and if applied too late, the result is "under cutting" or no cutting at all.

2. In dam construction, because of the considerable interval that must elapse between the time of clean up and the time of placing the next lift, contamination of the surface occurs. This may take the form of surface coatings of calcium carbonate or minute vegetable organisms, or it may be just plain dirt brought onto the concrete by workmen. At any rate it necessitates a second clean up just before the next lift is placed, and neither the air water jet nor wire brushing is likely to remove the contamination completely.

As a supplement to the air water jet both in the initial and final clean up at Boulder Dam there was employed a wet sand blast. As the air water jet was applied, the operator was trailed by a second workman with a bucket of sand. When an area was encountered for

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which the air water jet was insufficient to produce the necessary scouring action, one end of a short piece of rubber hose was thrust into the bucket of sand, the other end being connected to a third inlet to the air water nozzle. The sand was sucked up through the hose and there was produced a wet sand blast—a makeshift device but effective.

Not reported in the 1934 A.C.I. paper because it was felt that they were not sufficiently comprehensive to draw conclusions were some exploratory tests made at the University of California to determine the bond strength of construction joints where the hardened concrete of the old lift was lightly sand blasted but with no initial clean up with the air water jet. The results indicated that the sand blast applied to hardened concrete was approximately as effective in producing high bond strength as was the air water jet applied at that critical time when the surface layer of grout could be removed without dislodging pieces of coarse aggregate. Later the sand blast method was used for the construction joints between invert and arch for 55 miles of conduit sections of the Colorado River Aqueduct.

More recently comprehensive tests made by the Metropolitan Water District of Southern California and by the United States Bureau of Reclamation, both in the field and in the laboratory, have shown that the wet sand blast method is the equal of the air water jet method. It seems safe to assume that under ordinary construction conditions the wet sand blast method is considerably superior to the air water jet method, not only as regards the probability of securing the optimum in joint strength and impermeability but also as regards lower construction costs.

The wet sand blast method was specified for the first time on a large scale by the U. S. Bureau of Reclamation for the upper portion of Grand Coulee Dam, now being constructed by Consolidated Builders, Inc. Here the contractors have installed an elaborate sand blast system, the dry sand being stored in overhead bins from which it is discharged by gravity into pressure tanks distributed at intervals over the length of the dam. From these tanks the sand is fed through sand blast hose by compressed air to the nozzle. To the nozzle is also connected a water hose. The sand blast operator controls the water to suit conditions; the rate of sand feed is maintained constant by an orifice or by the setting of a valve in the bottom of the overhead pressure tank. This development has demonstrated on a large scale the practical possibilities of the single wet sand blast clean up, and it seems safe to say that its use on similar construction will become general.

Under the usual job conditions the surface of each lift is subject to the action of considerable foot traffic. Where, as with the air water jet the clean up is performed on a surface that has not yet hardened, there is always a fair chance that this traffic will disturb the bedment, and hence destroy the bond, of partially buried pieces of aggregate even though these pieces may not be dislodged. Since under the usual construction conditions a lift will be at least three days old before sand blasting, and hence will have acquired considerable strength, the possible adverse effect of foot traffic is remote.

Among a considerable number of those charged with the responsibility of dam construction the idea seems to prevail that to secure good bond the surface of the old concrete must be rough and irregular, with many pieces of coarse aggregate protruding from the mass. It is argued that this will increase the resistance to shear along the construction joint. To secure this condition it is common practice to undervibrate the top layer of each lift, resulting in incomplete compaction of this layer. Also, it is common practice during the clean up operation to cut away the mortar until the exposed surface is almost entirely composed of coarse aggregate.

The results of tests do not indicate the desirability of this procedure. On the contrary, where there is little or no water gain, as is the case with stiff mixes now used in dam construction, the indications are that the best possible bond is likely to be be secured when the concrete is compacted to a relatively smooth surface with all pieces of coarse aggregate completely embedded and when the clean up consists in barely removing the surface skin until pieces of fine aggregate are exposed. From a consideration of job operations one must conclude that a lightly cut, well compacted, smooth surface possesses definite advantages over a heavily cut, poorly compacted, rough surface, not only by reason of the economy involved, but more particularly because (1) pieces of the coarse aggregate at or near the surface are completely bonded to the underlying or surrounding concrete, (2) laitance and surface film may be more easily detected by the inspector, (3) the surface is more easily washed with less tendency for sediment to collect in low spots, (4) with a proper surface slope, the surface will more easily drain and such water as may collect in shallow depressions can be readily removed with an air jet, and (5) grout can be broomed into the surface more effectively.

BY R. F. BLANKS*

Mr. Wuerpel has introduced a subject which has claimed the increasing attention of engineers and constructors on important con-

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crete work during recent years. While the data presented by Mr. Wuerpel are admittedly rather meager, the information offers experimental proof of observations in the field regarding the behavior of concrete at lift planes and also substantiates the findings from extensive investigations of construction-joint clean-up methods conducted by the Bureau of Reclamation and other organizations.

Numerous examples can be found in existing structures which show that deterioration does not generally occur at a construction joint proper but rather in the concrete at the top of the lift, immediately below the joint plane. Also, observations and tests[1] of seepage along construction joints further show that "the inherent cause of such leakage is not necessarily the lack of bond and contact at the construction joint, but is more generally the result of a layer of porous low-quality concrete caused by segregation during the placing of wet mixes."[2]

A large number of tests have been conducted by the Bureau of Reclamation, both in the field and in the laboratory, on the bond strength at construction joints under various conditions of clean-up treatment, placing, compaction, and curing. The results of these tests, as well as information accumulated from other studies and construction experiences, cannot be presented in this discussion but it is anticipated that the data and information will be assembled for publication during the current year. In general, the bond tests show that any of the more commonly used methods of clean-up and placing will result in bond strengths at horizontal construction joints that are equal to or higher than the tensile strength of the concrete, provided the operations involved are properly conducted and controlled. The above statement applied for concrete of relatively high strength and quality.

Obviously the agreement between the above observations and the following statements in Mr. Wuerpel's paper is more than mere coincidence:

> Only one of the sixty-seven cores developed a joint failure, * * * Some of the cores initially frozen at an age of 14 days disintegrated completely after less than 70 cycles, without showing signs of joint distress prior to failure of the core. This specimen (Fig. 2) is representative of a group of cores extracted from a thin-wall section where the height of the lift to the joint plane was approximately 10 feet * * * The deterioration of the concrete below the construction joint is much greater than in the remaining portion of the core, but the joint, itself, remained sound.

Mr. Abrams, in his discussion of Mr. Wuerpel's paper, has made some statements and arrived at certain conclusions, apparently without

[1]Unpublished results of tests on permeability of construction joints conducted at Bull Run Dam, Oregon, by T. C. Powers, Portland Cement Association, Chicago, Ill., and by the Bureau of Reclamation, Denver, Colorado.

[2]Quotation from, Bureau of Reclamation, "Concrete Manual" Second Edition, 1939, page 209.

full knowledge of all the facts involved.* The following comments and information are presented in order that the readers may be more fully informed on some of the doubtful points raised by Mr. Abrams and so that fallacious interpretations and conclusions may be avoided.

In reviewing the history of the air-water jet method of initial clean-up for concrete construction joints, Mr. Abrams states:

> The method was invented by the contractors on Boulder Dam in 1933 as a labor saving device. It was substituted by the Bureau of Reclamation for the specified method of cleaning the *hardened* concrete by "* * * thorough scrubbing, brushing, chipping, sandblasting, washing, and blowing with compressed air (Specifications No. 519) * * *"

Specifications No. 519, issued in 1930, covering the construction of Boulder Dam contemplated the initial type of clean-up, that is, before the concrete had become thoroughly hardened, as is apparent from the following more complete quotation from these specifications:

> All concrete surfaces, such as the horizontal construction joints of the dam, * * * shall be roughened, and all laitance and loose or defective surface concrete shall be removed by thorough scrubbing, brushing, chipping, sand-blasting, washing, and blowing with compressed air. * * * The cleaning of concrete surfaces as described above *shall be done while the concrete is sufficiently soft* to permit thorough cleaning. Any surface which is *allowed to become thoroughly hardened* before cleaning shall be completely chipped to a depth as directed by the contracting officer.

The initial type of clean-up for horizontal construction joints, that is, removal of the surface film or layer before final set occurs, was, to the writer's knowledge, used at Exchequer Dam, California, in 1924 to 1926, at Pardee Dam, California, and Bull Run Dam, Oregon, in 1927 to 1929, at Owyhee Dam, Oregon, in 1930 to 1932, and at Morris Dam, California, in 1932 to 1934. Final clean-ups, that is, just prior to placing the succeeding lifts, were also used on these dams which varied from air-water washing to a combination of brooming, picking, and washing. In the beginning of the initial clean-up practice, cutting of the partially hardened concrete surfaces was usually accomplished with stiff wire brooms and brushes accompanied by either or both air and water jets. So far as the writer is aware, the exclusive use of high-velocity air-water jets in the initial clean-up treatment was first employed at Bull Run Dam. When decisions were being made as to the type of treatment to be employed in the initial clean-up at Boulder Dam, the advantages or disadvantages of the air-water jet as compared with wire brushing was a controversial question. Accordingly, the tests at the University of California, to which Mr. Wuerpel has referred and regarding which Professor Davis presents additional information

*Some critical passages in Mr. Abrams' discussion are directed at practices of the Bureau of Reclamation rather more than at Mr. Wuerpel's paper. It seemed proper that the Institute give the Bureau of Reclamation an opportunity to say something of those practices. This discussion by Mr. Blanks is the result.—EDITOR

in his present discussion, were conducted under a cooperative arrangement between Six Companies, Inc., contractors on Boulder Dam, the University of California, and the Bureau of Reclamation. As has been pointed out, the results of these tests indicated the superiority of air-water cutting as compared with wire brushing and the former method was officially approved for use at Boulder Dam. The air-water initial clean-up method proved generally satisfactory at Boulder Dam although it was found necessary to remove coatings and contamination from the joint surface areas by brooming or sand-blasting and washing in the final clean-up operation, immediately prior to placing the succeeding lift, as has been pointed out in Professor Davis' discussion.

Mr. Abrams refers to "The nonchalant manner in which this questionable method was adopted * * * " by the Bureau of Reclamation by citing a quotation from "The July 1936 edition of their Manual for the Control of Concrete Construction." It is pointed out that the *first edition* of the Bureau of Reclamation Concrete Manual was released in 1938. The "Tentative" volume to which Mr. Abrams refers was not an official publication of the Bureau in that it was not available for public distribution. It was intentionally incomplete with respect to many of the subjects treated in order to develop more effetively comment and information for the first official edition. Every effort was made to control the distribution of the "Tentative Issue" within the Bureau organization and to a few outside the Bureau who were in a position to offer worth-while suggestions and constructive criticism without misinterpreting the purpose of the book.

The results of the construction-joint tests at the University of California and the experiences at Boulder Dam in using the air-water jet method of initial clean-up with satisfactory results formed the basis for drafting the clean-up provisions in Specifications No. 570 for construction of the foundation portion of Grand Coulee Dam, issued in 1934, as follows:

> Concrete surfaces such as the horizontal construction joints of the dam * * * shall be prepared to receive the succeeding lift by having all laitance and loose or defective surface concrete removed by means of jets of air and water applied at high velocity and at the proper time during the setting period. If the clean-up is not performed at the proper stage in the hardening process and in such a manner as to render the surface suitable for bonding with new concrete, as determined by the contracting officer, the defective and undesirable concrete shall be removed by wet sandblasting * * *. The surface of each lift shall be washed immediately prior to the placing of the succeeding lift of concrete, to remove all foreign material which may have accumulated since the first clean-up operation: *Provided*, That where foreign material cannot be removed by washing, wet sandlbasting will be required.

The experiences in the use of the air-water jet initial clean-up during the first stage of construction at Grand Coulee Dam were not as favorable as at Boulder Dam. The job conditions, construction methods, character of the concrete, and the conditions and length of exposure of the construction-joint surfaces before being covered by the next lift at Grand Coulee Dam all contributed to increased contamination and surface coating as compared with conditions at Boulder Dam. Accordingly the use of wire brooms or sandblasting was resorted to in the final cleanup operation on many construction-joint surfaces in order to remove effectively, objectionable coatings and contamination. Sandblasting was required on something less than fifteen per cent of the area of such surfaces. The experiences at Grand Coulee led to the initiation of the extensive series of tests and investigations on construction joints by the Bureau as previously mentioned. Laboratory tests were conducted in Denver and at Grand Coulee and field tests at Parker, Seminoe, Marshall Ford, and Grand Coulee dams. Field and laboratory tests were also made by the Metropolitan Water District of Southern California and studies were carried out by the Tennessee Valley Authority at Hiwassee Dam.

Progress data and information from the above investigations were available by the time Specifications No. 757 for completing the construction of Grand Coulee Dam were issued in the latter part of 1937. These specifications accordingly definitely required a sandblast final clean-up with provisions whereby, in addition, an initial clean-up could be required or omitted at the discretion of the contracting officer. Some attention was also given to the condition in which the surface should be left upon completing a concrete lift in the last Coulee Dam specifications. More complete information was available when Specifications No. 780 for the construction of Shasta Dam in California were issued in 1938 and further progress was possible in drafting specification requirements for construction-joint treatment to insure the desired results at minimum cost. Specifications for important work soon to be issued by the Bureau of Reclamation will provide for construction-joint treatment about as follows:

> Surface vibrators or puddlers shall not be used. Excessive vibration sufficient to cause segregation and laitance or tending to bring an excessive amount of water to the surface shall be avoided. In compacting the surface of a concrete lift the coarser particles of aggregate on the surface shall be completely embedded by booting and shoveling methods, while the concrete is being vibrated, but excessive working of the concrete such as will result in an unnecessarily smooth surface shall be avoided. Embedment of coarse aggregate after vibration is completed will not be permitted. Disturbance of the surface concrete at a construction joint during the early stages of hardening shall be avoided and no unnecessary traffic will be permitted on new concrete until it has hardened sufficiently to withstand such treatment without

injury. Surfaces of construction joints which have been permitted to dry by reason of the succeeding lift or adjoining concrete not being placed within the specified moist-curing period shall be moistened and kept continuously moist for at least 72 hours immediately prior to placing the succeeding lift or adjoining concrete. Immediately before placing concrete in the succeeding lift, the entire surface of the construction-joint shall be thoroughly cleaned of all loose, defective, and fractured concrete, and of all laitance, coatings, stains, debris, and other foreign material, by wet sandblasting, as directed by the contracting officer. The surface of the concrete shall then be washed to remove all loose material. All pools of water shall be removed from the surfaces of construction joints before new concrete is placed. Construction joints shall be covered with a layer of mortar approximately one-half of an inch thick. The mortar shall be spread uniformly and shall be worked thoroughly with stiff brooms into all irregularities of the surface. Concrete shall then be placed immediately upon the fresh mortar.

Professor Davis in his discussion of Mr. Wuerpel's paper mentions the prevailing idea among some engineers that "the surface of the old concrete must be rough and irregular, with many pieces of coarse aggregate protruding from the mass." It is entirely possible that certain provisions in specifications and statements in the "Concrete Manual" recently issued by the Bureau of Reclamation are subject to such interpretation. These provisions and statements were intended in part for the purpose of preventing or discouraging the practice of puddling and tamping the surface after a concrete lift is completed to embed particles of coarse aggregate and secure an overly smooth surface to facilitate clean-up procedure. It is hoped that the specification provisions quoted in the preceding paragraph will correct the misinterpretation.

The preceding paragraphs provide a chronological record of developments and progress in construction-joint treatment in dams from 1924 to the present. The Bureau of Reclamation still employs the air-water jet initial clean-up for some types of work under certain conditions and does not condemn the method in its "Concrete Manual" as Mr. Abrams has claimed. In order that the readers may have all the facts of the case from which to draw their own conclusions, section 102, pages 208 to 214, of the Bureau of Reclamation "Concrete Manual," Second Edition, dealing with construction joints, is quoted verbatim as follows:

102. *Construction Joints.*—Recent Bureau specifications define construction joints as concrete surfaces which have become so rigid by reason of limitations or delays in construction progress that new concrete cannot be integrally incorporated with that previously placed. Ordinarily uniformed construction joints in concrete are horizontal or approximately so.

The character of the concrete and the placing methods have an important effect on the surface condition of a completed horizontal joint. Mixes that are too high in slump, that segregate and bleed badly or that otherwise tend toward the formation

of excessive laitance at the surface should be particularly avoided. Excessive working of surface concrete and traffic over the surface before the concrete hardens are factors contributing toward undesirable conditions. Necessary traffic in setting anchor bolts and tie wires or in placing embedded instruments should be permitted only on planking that should be removed as soon as the operations are completed. Protrusion of some coarse particles at a construction joint surface is considered desirable, hence tamping or other working to secure complete embedment should not be permitted. It is comparatively easy to secure a high degree of bonding and water-tightness at a concrete construction joint that is reasonably smooth, hard, and free from laitance or coatings. Unfortunately it is not always practicable to obtain such surface conditions because of the character of materials available for use in the concrete, accumulations of debris and coatings of various kinds from construction processes, or other reasons. Treatment of the joint surface preparatory to placing new concrete, commonly termed "cleanup", is accordingly required to insure desired bonding and watertightness. The extent of the preparatory treatment required may range from simple washing with air-water jets for ideal surface conditions to the removal of an appreciable amount of surface material where damaged or otherwise defective concrete exists.

Various schemes for the preparation of horizontal construction joints have been evolved in past years to insure bonding and watertightness. Such provisions were generally occasioned by the use of concrete of very wet consistency and the chuting method of placement. The preparatory measures employed have in some instances been ineffective, as the evidence of seepage at *fill planes* in existing structures will testify. The inherent cause of such leakage is not necessarily the lack of bond and contact at the construction joint, but is more generally the result of a layer of porous low-quality concrete caused by segregation during the placing of wet mixes. This is borne out by the observation that the leakage in many instances does not appear directly at the construction-joint plane but immediately below it. Fig. 68, a photograph of concrete placed in a hydraulic structure 25 years ago, illustrates segregation resulting from the use of too-wet concrete and emphasizes the necessity for giving proper attention to the character of concrete and its manupilation in placing.

The so-called "initial cleanup" method of horizontal construction joint treatment has been employed as more or less standard procedure on certain types of Bureau structures during recent years. The initial cleanup consists of cutting the surface of concrete, usually with a high velocity air-water jet, removing the surface layer, and exposing the coarse aggregate particles before the concrete attains its final set. The exact time for performing an initial cleanup operation depends upon the character of the concrete, the type of cement used and the atmospheric conditions with respect to temperature and humidity. Under some conditions the opportune time is quite critical and may range from six to twelve hours or more after placing. When the initial cleanup is employed, constant vigilance is necessary to avoid undesirable results caused by: (a) the use of a jet of too high intensity, (b) the removal of excessive quantities of sound concrete, (c) disturbance of the surface aggregate particles, and (d) performing the operation too early or too late. In general the cutting should be quite light rather than deep and just sufficient to remove any laitance and the surface film. In some instances the cutting for the initial cleanup has been accomplished by using stiff wire brooms and removing the cuttings by air and/or water jets after they have dried. This procedure is even more critical with respect to time than the air-water-jet method and requires more care in manipulation to secure the desired results.

FIG. 68—FROM BUREAU OF RECLAMATION "CONCRETE MANUAL" SHOWING SEGREGATION IN OVER-WET CONCRETE AT FILL PLANES

Satisfactory results can be obtained with the initial cleanup treatment if it is properly performed and controlled. However, during the interval of time before the succeeding lift is placed, the surface of the old concrete, unless suitably covered, generally becomes contaminated, acquires various types of coatings, or is damaged. As a result, a second or "final cleanup" is required to properly prepare the joint for placing new concrete. The combined cost of the initial and final cleanups becomes an appreciable item in the total cost of the concrete manufacturing and placing operations. Furthermore, recent observations and investigations indicate that the disturbance involved in initial cleanup procedure may injure the freshly placed concrete. In other words, there seems to be a critical period in the concrete hardening process, apparently between six and forty-eight hours, during which the concrete is subject to damage if disturbed. It is not always practicable to avoid conflict between this critical period and the optimum time for performing the initial cleanup. As a consequence of the considerations and experiences just discussed and exhaustive investigations of cleanup methods during the past few years, the Bureau is omitting the initial cleanup on important work and employing only a final cleanup treatment for horizontal concrete construction joints. Two recent Bureau specifications for large dams have stipulated this latter method.

The final cleanup of horizontal construction joints, whether performed in conjunction with an initial treatment or employed alone as a preparatory measure, can be satisfactorily accomplished in several ways. For ideal surface concrete which is hard, dense and free from laitance or coatings, a thorough air-water-jet washing may be the only treatment required. The most positive means of insuring complete removal of all undesirable surface material in the final cleanup operation is the use of a wet sandblast followed by water washing. Discretion is necessary, however, in wet-sandblasting, at least for ages up to about seven days when low-heat cement is

used, in order to avoid overcutting and permanent disruption of bond between some of the coarse aggregate particles and the cementing matrix. If the concrete has not become too hard, satisfactory results may be obtained in the final cleanup by thorough wire brushing or picking with hand picks, followed by water washing. Chipping with jackhammers should be employed only where absolutely necessary to remove damaged or defective material because of the danger of permanent injury to the concrete.

Attention should be called to the important influence of the curing treatment employed at horizontal construction joint surfaces on the required cleanup treatment. Evaporation of curing water may deposit calcium hydroxine leached from the concrete which subsequently forms a coating of calcium carbonate. The curing water may also carry cuttings or foreign substances which are deposited as coatings on the joint surface. Premature drying of the surface concrete may result in a soft, porous layer. For these reasons, a recent Bureau specification requires the use of a substantial layer of moist sand for curing construction joint concrete in order to avoid or minimize the conditions mentioned.

Whatever cleanup method may be used for a particular job, it is very important that a mortar layer of the proper quality and consistency be thoroughly broomed into all the irregularities and depressions of the joint surface immediately prior to placing new concrete. This mortar layer should be about one-half inch thick, should have a water-cement ratio not in excess of that used in the regular concrete and should have cement-to-sand proportions that will result in a slump of about six to eight inches. Mortar that is too dry cannot be effectively broomed into the smaller depressions, and mortar that is too wet tends to segregate. Dry concrete surfaces should be kept wet for a period of at least 72 hours before new concrete is to be placed but no pools of water should be permitted on the surface when the mortar layer is placed.

Formed construction joints should be avoided wherever practicable as they are inherent planes of weakness that are susceptible to the formation of cracks and passage of water unless suitable water stops are provided. The surfaces of formed construction joints should be roughened and the surface film removed before new concrete is placed. This may be accomplished effectively by wet-sandblasting or, if the concrete is not too hard, by hand picking and wire brushing, followed by thorough washing. In placing new concrete against a formed construction joint, particular care should be exercised to insure intimate contact and to fill all irregularities and depressions by careful spading and puddling with suitable tools. Where practicable the surface should be scrubbed with a wire broom dipped in the fresh concrete immediately before it is covered.

No fixed rules applicable to all jobs can be laid down for the treatment of construction joints preparatory to placing new concrete. The process of combination of processes must depend upon the circumstances and individual conditions obtaining for each job and the judgment of those in charge of the work.

Mr. Abrams has criticised the Bureau's "million-dollar research department" for delay in determining the facts in connection with construction-joint treatment. Strictly speaking, the Bureau of Reclamation has no research department. The laboratory facilities maintained by the Bureau are employed as design and construction control tools.[3] While every effort is made in the Bureau's investiga-

[3] *Engineering News-Record*, March 30, 1939, pages 446 to 450.

tional work to solve the engineering problems as they arise, it is extremely difficult to close entirely the "lag between research and practice" as Frank T. Sheets has so aptly inferred.[4]

Acknowledgments. All activities of the Bureau of Reclamation are supervised by John C. Page, Commissioner, Washington, D. C. and all engineering and construction are under the direction of R. F. Walter, Chief Engineer, and S. O. Harper, Assistant Chief Engineer, with headquarters in Denver, Colorado. Research and design work are directed by J. L. Savage, Chief Designing Engineer and W. H. Nalder, Assistant Chief Designing Engineer. Arthur Ruettgers, Senior Engineer, and the writer head the Materials, Testing, and Control Division of the Denver organization.

AUTHORS' CLOSURE

The discussion evoked by my article on horizontal construction joints is gratifying in view of the importance of the subject and the need for general dissemination of up-to-date data on the matter of joint preparation.

The tests reported were not conceived to cover fully the subject of construction joint preparation, but were reported as a supplementary contribution to the work done by the University of California and other agencies.

The compressive strength data shown in the table of basic data in the article are not subject to the re-arrangement Mr. Abrams has indicated in his discussion. The water-cement ratios, cement factors, types of cement and aggregate and the mixture consistencies were different in the concrete entering the various sections tested. The compression tests made were too few in number to be used as a criterion of joint quality and were reported simply as general information on the types of concrete used in the work.

The cores tested in compression contained joints midway in their height. The load was applied in a direction normal to the joint planes. The ruptured cores indicated the usual "double cone" failure with no evidence of weakness at the joints.

***The importance of this subject prompts extension of the discussion period to July 20 for the September* Supplement.**

[4]Address by Frank T. Sheets before the 34th Annual Convention of the American Concrete Institute, Chicago, February 1938; JOURNAL, Amer. Concrete Inst., May-June 1938; *Proceedings* Vol. 34, p. 541.

Vol. 35 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *JANUARY 1939*

Corner Effects in Rigid Frames*

By Joseph A. Wise†

SYNOPSIS

In the analysis of reinforced concrete rigid frame structures, centerline distances are used for span lengths and corrections are applied to the moments thus found to determine the moments in beams, at edge of columns. This paper merely outlines the problem and one tentative solution which has been offered and is intended to provoke further discussion.

In the analysis of rigid frames there are certain problems connected with the corners, or intersections of members that have not been thoroughly investigated. The behaviour of such corners under test has been ably presented by Richart and Olson.[1] The relation of results obtained by them and by others to our methods of analysis needs to be considered.

In the first place it is quite clear that the common theory of flexure does not hold at such a corner. Even though we might assume Hooke's Law as valid, the stress distribution is no longer linear at a section close to the corner. However, we have no way of modifying our various methods of analysis which are all based on the validity of the common theory of flexure, to take into account this departure from the theory in the corners. Apparently it is necessary to make some assumptions as to behaviour which will give results as close as possible to those indicated by tests.

In order to make the problem clear, a simple example was chosen for analysis, as shown in Fig. 1.

*Received by the Institute August 24, 1938.
†Associate Professor of Structural Engineering, University of Minnesota, Minneapolis.
[1]Rapid and long time tests on Concrete Knee Frames. Journal Amer. Concrete Inst., March-April 1937; *Proceedings* Vol. 33, p. 459.

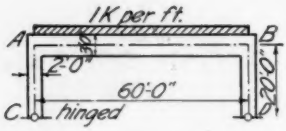

FIG. 1

If this frame is analyzed by any method, assuming members to be geometrical lines, as shown in Fig. 2.

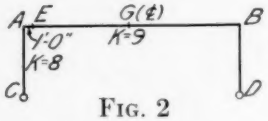

FIG. 2

$$M_{BA} = M_{AB} = -171 \text{ ft. kips}$$
$$M_G = +129 \text{ ft. kips}$$
$$M_E = -142 \text{ ft. kips}$$

If we assume that the moment of inertia of the beam AB varies at the end, using an arbitrary law of variation, $I = I_0\left(\frac{a}{x}\right)^3$ as indicated in

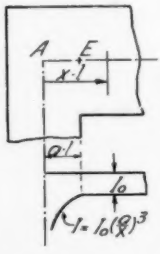

FIG. 3

and a similar law at top of columns, we will approximate at least roughly, the behaviour of the member. In addition, the reaction at top of column will not be a single concentrated force, but will be distributed in some manner similar to Fig. 4.

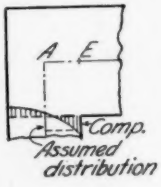

FIG. 4

Since this distribution may vary widely we assume an approximate, and equivalent uniform distribution as shown by the dotted line in Fig. 4. We now find,

$$M_{AB} = -171 \text{ ft. kips}$$
$$M_E = -157 \text{ ft. kips}$$
$$M_G = +279 \text{ ft. kips}$$

This varies appreciably from the first result, and one conclusion which could be drawn from it, is that the effect of rigidity of a corner, and the effect of distribution of loads on supports, is to increase M_E. In the pamphlet, "Continuity in Concrete Building Frames" (Second Edition), issued by the Portland Cement Association, it is recommended that a reduction of $1/3\ V\ (2a)$ be used to obtain the moment at edge of column. This would give, in this case, $-171 + 1/3 \times 29 \times 2 = -152$ ft. kips, which is close to the value -157 kips, obtained by the second analysis. Numerous other investigations have indicated that this approximation is generally close to that obtained by various assumptions of varying I at ends of members and various assumed distributions of reactions. However, for frames with wide members further investigation is necessary.

The use of clear spans is not consistent with the equations of statical equilibrium unless a rather complex correction is made, as indicated in Fig. 5.

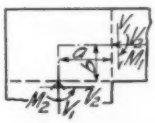

FIG. 5

$$M_2 = M_1 + V_1\, a - V_2\, b$$

Unless $V_1\, a = V_2\, b$, which is seldom true, M_2 will not equal M_1.

The tentative conclusion drawn from studies made to date, is that a method of correcting for corner effects in rigid frames is necessary; and that the Portland Cement Association approximation for moment at edge of column is good for usual frames. The use of clear spans is not valid unless complex corrections are introduced into the analysis.

Discussion, to close in June 1939 JOURNAL, should reach A. C. I. Secretary in triplicate by Apr. 1.

JOURNAL
of the
AMERICAN CONCRETE
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7400 SECOND BOULEVARD, DETROIT, MICHIGAN JUNE 1939

Discussion of a paper by Joseph H. Wise

Corner Effects in Rigid Frames*

BY E. MIRABELLI, L. B. ALMY AND CLOSURE BY THE AUTHOR

ERRATA

In the original paper p. 190 line 4 should read: $M_G = +279$ ft. *kips;* p. 191, line 6, should read: $M_G = +264$ ft. *kips.*

BY E. MIRABELLI

The writer agrees with Professor Wise's conclusion that the bending moments found in the analysis of a reinforced concrete frame using centerline distances should be modified in some way to allow for the effect of the corners or joints. The principal difficulty in making the modification is in evaluating the extent to which distortion of the material in the joint prisms contributes to the distribution of moments. If the joint material is assumed to be without distortion, that is to possess perfect rigidity, then the bending moments may be found by using clear beam spans and clear column heights. Under vertical loading, the amounts of the column moments and the beam negative moments thus found will, in general, be higher than the true amounts because the frame is assumed more rigid than it is actually. The actual moments probably have amounts somewhere between those found by using clear distances and those found by using center line distances.

If it may be assumed that the use of clear distances involves no appreciable change in the degree of joint restraints, then an approximate method may be developed for correcting for joint effect. This assumption amounts to stating that the moment factors for clear spans are substantially the same as for center line spans. That the moment

*JOURNAL, Amer. Concrete Inst., Jan. 1939; *Proceedings* Vol. 35, p. 189.
†Associate Professor of Civil Engineering, Massachusetts Institute of Technology, Cambridge, Mass.

factors are not necessarily the same is realized when it is observed that the elastic line from joint intersection to face of column (points A and E, Fig. 6 (a) does not necessarily remain straight under loading.

Referring to Fig. 6 (b) and letting the joint restraint be represented by the moment factor C, the symmetrically loaded beam, AB, will be subjected to end moments $M_A = -CwL^2$ based on center line distance or $M_E = -Ck^2wL^2$ based on clear distance. For a uniformly loaded beam, then, the bending moment, M_E, at the face of column may be obtained from the center line moment, M_A, by multiplying by k^2, the square of the ratio of clear span to center line span. This method is somewhat simpler than the Portland Cement Association Method in that it does not involve shear.

A similar procedure applied to the case of a beam loaded with a single concentrated load at mid-span, FIG. 6 (c) indicates that $M_E = kM_A$. In the case of wind moments, Fig. 6 (d), indicates that there is no joint effect and that consequently $M_E = kM_A$.

In the absence of "corner effect" the moment at the face of column for a uniformly loaded beam may be shown to be $M_{E1} = \left(\frac{1}{8} - C - \frac{k^2}{8}\right)$

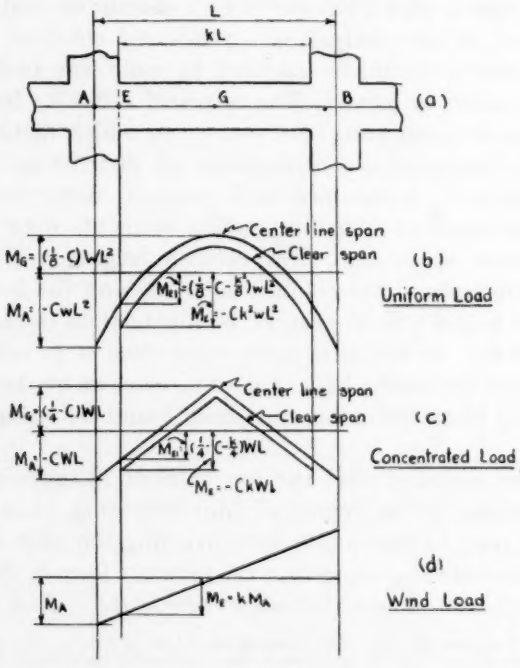

FIG. 6

$w L^2$, Fig. 6(b). Comparing M_{E1} and M_E it is found that

(a) When $C < \frac{1}{8}$, $M_E > M_{E1}$,
(b) When $C = \frac{1}{8}$, $M_E = M_{E1}$,
(c) When $C > \frac{1}{8}$, $M_E < M_{E1}$.

Condition (c) is applicable when the bending is negative over the entire length of beam. Under such a condition the effect of rigidity of joint is to reduce the bending moment at E rather than increase it. A case of this type is illustrated by the frame shown in Fig. 7, for which the following results are found.

Using center line distances: $M_{AB} = -1028$ ft. kips, $M_E = -998$ ft. kips.

Using clear distances: $M_E = -922$ ft. kips

Using Portland Cement Assoc. Method: $M_E = -1028 + \left[\frac{29x2}{3}\right] = -1009$ ft. kips

Using method suggested above: $M_E = \left[\frac{58}{60}\right]^2 \times 1028 = -962$ ft. kips

The actual amount of M_E is somewhere between $M_E = -998$ ft. kips and $M_E = -922$ ft. kips, the latter figure being based on the assumption of absolute rigidity of joint material.

In the usual frame, however, condition (a) is applicable and the effect is to increase the moment at E.

With a single concentrated load at mid-span, Fig. 6(c) it is found that

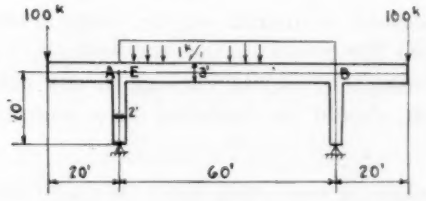

FIG. 7

(a) When $C < \frac{1}{4}$, $M_E > M_{E1}$,
(b) When $C = \frac{1}{4}$, $M_E = M_{E1}$,
(c) When $C > \frac{1}{4}$, $M_E < M_{E1}$.

The preceding comments apply to beams. Columns in vertically loaded structures seem to be affected by the joints to a greater extent than beams. A number of cases tested by the writer appear to indicate that the bending moment in the column at a section at the base

of the beam may be larger than that found at the intersection of center lines. For example, in the author's illustration (Fig. 1, p. 190) the column moment consistent with the beam moment $M_E = -157$ ft. kips (.p 191) is $M_{col.} = 172.5$ ft. kips, which is larger than the center line moment $M_A = 171$ ft. kips. The actual column moment is a function of the joint restraint, the ratio of clear beam span to center line span, and the ratio of clear column height to center line height, and is expressed by an equation too complicated for use in rapid design. The writer suggests using a moment for column design equal to the theoretical column moment at the intersection of center lines.

BY L. B. ALMY*

Mr. Wise suggests a method of attack for a more accurate solution for the moment at the face of the column in a rigid frame. With the same frame as Mr. Wise used in his example, the writer has computed moments for beam depths and column widths varying from 1 to 4 ft. Starting with the conventional assumptions of beams and columns concentrated at their center lines, three corrective assumptions were made.

The first consideration was of the effect of the change in moment of inertia of the beam between the face and the center line of the column and the similar change in moment of inertia of the column between the lower face and the center line of the beam. This moment of inertia was assumed to vary as $\left(\frac{a}{x}\right)^3 I_o$ where 'a' is half the column width, I_o, the moment of inertia of the beam between columns and x is measured from the center line of the column. No investigation was made on moments of inertia varying in any other way although such investigation should be included in a complete study of the situation.

In the second place, a correction must be made for the distribution of the vertical thrust over the width of the column. Fig. 8(a) shows the forces and moments in the conventional method of solution. In Fig. 8(b) two new equal and opposite forces have been introduced. Retaining the span length as the distance between center lines of vertical members, the result of these forces is an upward distributed load of intensity,

$$w = \frac{W\,l}{2d}$$

*Teaching Assistant, University of Minn., Minneapolis.

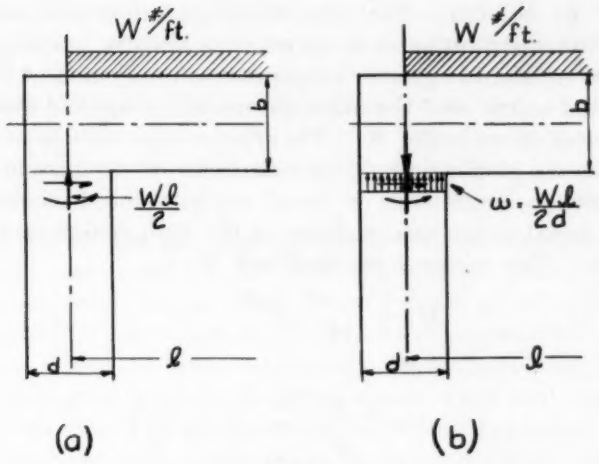

FIG. 8

distributed over a distance of $\frac{d}{2}$ and a concentrated external load, of intensity $\frac{w\,l,}{4}$ and acting upward at a point $\frac{d}{4}$ from the center line. From the effect of the uniform load on the beam, the distributed upward load over half of the width of the column and the external load applied to the joint, and considering the beam having the varying moment of inertia as described above, the moment may be determined at the intersection of the neutral axis. This moment will be designated hereafter as M_o.

Finally the moment in the rigid corner cannot be resisted entirely at the neutral axis, but must be distributed in some manner over the

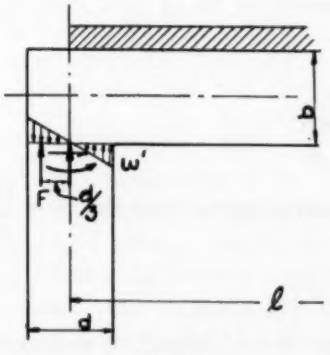

FIG. 9

width of the column. For ease of computation and satisfactory accuracy this distribution shall be taken as linearly varying. Introducing two equal and opposite moments into the system of Fig. 8, one a distributed couple and the other an externally applied moment, we have forces as shown in Fig. 9. The effect of thes additional moments is to reduce the moment M_o. The magnitude of the introduced equal and opposite moments must be equal to the reduced moment of the joint, i.e., equal to the final moment of the joint considering all three corrections. This moment we shall call M_t

$$M_t = \frac{w_1 d^2}{6} = \frac{Fd}{3}$$

and

$$w^1 = \frac{6 M_t}{d^2} \quad \text{or} \quad F = \frac{3M_t}{d}$$

That part of the distributed moment outside the center line may be considered an externally applied moment being opposite in direction to F and half the magnitude of F. The resultant loads then are a linearly varying upward load of intensity zero at the center line and intensity w^1 at a distance $\frac{d}{2}$ from the center line and an upward external force of magnitude $\frac{w^1 d}{4}$ acting at a point $\frac{d}{3}$ outside the center line. The effect of these loads is to produce a moment of magnitude Rw^1 in the opposite direction to M_o, where R is a determinable constant depending upon the distributed external moment and the moment in a beam of varying moment of inertia loaded with the triangular upward load. Then we may say:

$$M_t = M_o - w^1 R = M_o - \frac{6M_t}{d^2} x\ R$$

or

$$M_t = \frac{M_o}{1 + \frac{6R}{d^2}}$$

We find in every case investigated that the term $\frac{6R}{d^2}$ was equal to $\frac{1}{2}$ or

$$M_t = \frac{2}{3} M_o$$

Taking into account these corrective assumptions, moments M_o, M_t and M_{tc}, the moment at the face of the column, were computed

for twelve different cases of varying depths and width of beam and column. Table I records the results of these computations. $M.$ is the moment at the intersection of the center lines computed in the conventional method. The last column is the moment at the face of the column as computed by the method suggested in the pamphlet "Continuity in Concrete Structures" (Second Edition) issued by the Portland Cement Association.

The results of this investigation show that for rigid corners the reduction recommended by the Portland Cement Association gives close approximation for the depths of the beam equal to or greater than the width of the column. The error in these cases is all within 3 per cent. When the beam depth is less than the column width the error becomes greater reaching approximately 8 per cent in the worst case investigated. The average error for the 12 cases was —1.6 per cent and the mean deviation from the mean 3.9 per cent. This would indicate then, in the majority of the cases, the error involved in reducing the moment at the face of the column by $\frac{1}{3}$ the shear at the face of the column would be less than 5.5 per cent. Furthermore that with the beam depth greater than the column width, the correction is sufficiently accurate but for beam depth less than column width for accurate results some method of calculation of the moment at the face of the column should be used other than a simple shear correction.

TABLE 1

Case	d	b	Ms	Mo	Mt	Mfc	$Ms-\frac{1}{3}\sqrt{d}$
I	2	3	171.5	179.6	119.7	157.6	152.5
II	2	2	245.4	248.2	165.3	226.0	226.1
III	3	2	281.4	281.3	187.7	248.8	252.9
IV	3	3	245.4	249.7	166.6	216.3	216.9
V	1	2	108.0	112.9	75.2	101.7	98.2
VI	1	4	19.7	21.0	14.0	9.9	9.9
VII	2	1	291.9	290.0	193.3	268.0	272.6
VIII	2	4	108.0	118.3	78.9	96.3	88.7
IX	3	1	297.6	296.9	198.0	264.3	269.1
X	4	1	299.6	298.9	199.3	256.1	262.3
XI	1	3	42.9	46.0	30.7	34.9	33.1
XII	1.5	3	108.0	115.5	77.0	98.9	93.9

AUTHOR'S CLOSURE

Professor Mirabelli's statement that if the joints are assumed to possess perfect rigidity, then the bending moments may be found by using clear beam spans and column heights is either incorrect or is not clearly expressed. Using clear span and column height in the example in his duscussion, that is, using cantilever arms of 19 feet, center span 58 ft. and column height 18.5 ft., I find the moment at end of beam AB (at point E) to be 958 ft. kips. However, using center

to center spans but assuming joints rigid, this moment is 922 ft. kips. Furthermore, if we apply the suggested correction factor, $\left(\frac{58}{60}\right)^2$, to the moment due to the uniform load only, as its derivation indicates should be the case, and applying no correction factor to the moments due to loads on cantilevers, since they produce a constant negative moment in center span, I find that the moment at E would be 1018 ft. kips. Thus the suggested correction factor, if applied logically, is not even as good as the P.C.A. method of correction, and it is admitted that in this case the P.C.A. method undoubtedly overestimates the moment. The method suggested by Professor Mirabelli appears to give a fairly good result only because the loads on the cantilever arms are moderately large. If they had been chosen very much smaller or very much larger, the correction would not be as good. A correction based on shear appears much more logical and more likely to be generally valid. The tendency for column moments to be higher when the joint prisms are considered rigid is due to the fact that, since the columns are usually shorter than the beams, their total rigidity is increased proportionately more than that of the beams when the results for rigid joints are compared with those for centerline spans. It would seem desirable, therefore, to have the method of correction take this fact into account.

The question is one which should receive more study.

Mr. Almy's discussion indicates the desirability of extending the analysis of the problem to more complex cases. His work has been very thorough and he is to be congratulated for his clear presentation of the results.

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *JANUARY, 1939*

Reinforced Concrete Girder Bridges of Over 100 Ft. Span*

By K. Hajnal-Kónyi†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

Prompted by Professor Pletta's paper on rigid frame bridges in the United States, the present author tabulates 105 bridges in 17 countries.

The writer has recently published a list of 97 recorded reinforced concrete girder bridges of over 100 ft. span with solid webs including the longitudinal and cross section of these bridges and their main dimensions.[1] Since this publication he has obtained knowledge of 8 other bridges of this description. In the following 4 tables an analysis of these bridges will be given similar to that in D. H. Pletta's paper on rigid frame bridges.[2]

A few preliminary remarks will explain the classification of the bridges which is based on a new definition of a girder bridge, introduced by the writer in 1933. The conventional definition of a beam is a structure in which only vertical reactions occur due to vertical loads; structures with a horizontal thrust induced by vertical loads are not considered as beams. The application of this principle results in certain difficulties as some structures, the very nature of which is arch-like, must be called "beams" and vice versa. For this reason the writer suggested another definition of the beam,[3] based on the bending moment diagram between two supports. Continuous structures over more than one span are regarded as subdivided into single spans by verticals through the centre of the supports, and each span is considered separately. There are continuous structures composed of

*Received by the Institute Aug. 23, 1938.
†Consulting Engineer, London, England.
[1] *Concrete and Constructional Engineering*, Vol. XXXIII, Nos. 1, 2, 4, and 5, 1938.
[2] "*Notes on Existing Rigid Frame Bridges in the United States*, JOURNAL, Amer. Concrete Inst., March-April, 1938. *Proceedings*, Vol. 34, p. 517.
Beton u. Eisen, 1933, No. 23, 24.

various systems with beams in some spans and arches in others. The definition is as follows: Beams are structures whose bending moment diagram can be derived from that of a simply-supported beam by adding a trapezium (in limiting cases a triangle or a rectangle). This definition also includes intermediate spans of continuous girders with hinges (Fig. 1, 2, 3).

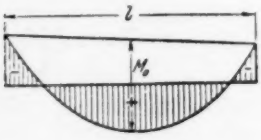

FIG. 1.

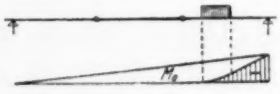

FIG. 2.

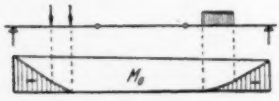

FIG. 3.

Courtesy *Concrete and Constructional Engineering*

The mathematical expression of this definition is that the bending moment on a beam due to a vertical load at a distance x from the left-hand support is

$$M = M_o + \frac{l - x}{l} M_e + \frac{x}{l} M_r$$

in which M_o denotes the bending moment in the simply-supported beam between any two adjacent supports, M_e and M_r are the bending moments at the left- and right-hand supports of the span under consideration.

In order to emphasize the difference between beams and arches or arch-like structures, it may be added that the bending moments in arches can be derived from that of a simply-supported beam by adding a closing line composed of at least two straight lines, a polygon or a curved line. The mathematical formula for the bending moment in arches and archlike structures is

$$M = M_o + \frac{l - x}{l} M_l + \frac{x}{l} M_r - H\Delta y$$

It contains the additional term $-H\Delta y$. This must disappear if the structure is to be called a beam. H denotes a thrust, whereas Δy is the ordinate of the axis of the structure measured from the straight

line connecting the intersections of the axis with the verticals at the supports ($\Delta y = 0$ at $x = 0$ and $x = l$). It is obvious that $H\Delta y$, being a product, will disappear if either $H = 0$ or $\Delta y = 0$. The first condition is static; the second is geometric.

We see, therefore, that according to the new definition a structure can be a beam, even if H is not zero, provided $\Delta y = 0$. For the structure itself between two supports it is of no significance whether H is an external or internal thrust taken by the abutments or by a tie.

Briefly, all structures should be considered as beams in which the bending moments between two adjacent supports differ from the bending moments of a simply-supported beam (between the same supports) by the effect of the moments at the supports only. These support moments can have any value; they depend on the fixing, if any, at the supports, the influence of the loads on, and connections to, adjacent spans and cantilevers, or on the settlement or movement of the supports.

A closer investigation of the characteristic features of girder bridges is necessary in order to arrive at a classification which satisfactorily points out their structural relation.

The main difficulty in the design of large girder bridges is that of dealing with the positive bending moments in the span. With increase in the span the influence of the dead load also increases so that above a certain limit a bigger section does not increase the bearing capacity of the bridge. It is easier to deal with support moments, since an increase of the dimensions at the supports does not effect the bending moments very much. With regard to the depth of the structure at the supports the designer has usually more freedom than in the span. Therefore when designing large girders in reinforced concrete it is essential in most cases to keep the positive bending moments as small as possible. There are many ways of doing this.

In a continuous beam with nearly equal spans a reduction of the span moment is automatically obtained, and in consequence the continuous beam is the most natural and most frequently adopted form of reinforced concrete girder bridges with large span. In this system many variations can be made, for instance by arranging hinges in order to obtain statically determinate system and keep the zero points fixed independently of the position of external loads, movements of supports, etc.

Consideration of the different methods of reducing the span moments leads to types of girder bridges (Fig. 4) as follows:

(1) Simply supported beam, fixed in position at one end and capable of horizontal movement at the other.

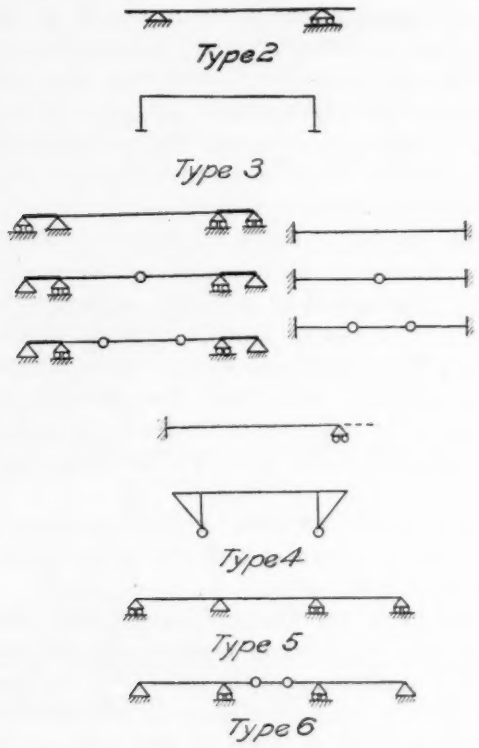

FIG. 4. TYPES OF GIRDER BRIDGES
Courtesy *Concrete and Constructional Engineering*

(2) Simply supported beam with cantilevers. Cantilever moments are induced by dead load as required for the relief of the span moments.

(3) Rigid frames with horizontal top. (This type can be considered as a special case of type 4.)

(4) Beams partly or totally fixed by adjacent short spans or by large abutments.

(5) Continuous beams without hinges.

(6) Continuous beams with hinges.

Table 1 contains 105 bridges in 17 countries. Each bridge has been mentioned in that country under the authority of which it was executed disregarding later political alterations.

The first country to build reinforced concrete girder bridges of over 100 ft. span was Hungary where such bridges were completed 30 years ago. Then followed Germany (1911), Switzerland (1913), Austria (1917), U.S.A. (1919). In Hungary a series of bridges of a

TABLE 1—NUMBER OF LARGE REINFORCED CONCRETE GIRDER BRIDGES WITH SOLID WEB IN VARIOUS COUNTRIES

COUNTRY	TYPE						Total number in each country	Year of completion of the first bridge in each country	Maximum span in each country	
	1 Simply-supported	2 Simply-supported with Cantilever	3 Rigid frames with horizontal top	4 One comparatively large span partly or totally fixed	5 Continuous beams without hinges	6 Continuous beams with hinges			ft.	in.
Albania	....	1*	....	....	....	....	1	1931	121	5
Austria	....	....	....	....	....	3	3	1917	110	11
Belgium	....	....	....	....	2	2	4	1927	182	1
Brazil	....	....	....	....	2*	....	2	1930	223	1
Canada	....	....	....	1	....	....	1	1924	209	10
Czechoslovakia	1	....	....	1	....	3	5	1928	172	3
France, incl. colonies	1*	....	....	1*	2	1	5	1930	232	1
Germany	....	3	3*	7	3	17*	33	1911	226	4
Great Britain	1	....	....	....	2	1	4	1935	160	0
Hungary	....	2	....	13	2	1	18	1908	126	1
Irish Free State	....	....	....	1	....	2	3	1928	125	6
Italy	....	2	....	....	1	1	4	1928	170	7
Japan	....	....	....	....	....	6	6	1934	113	2
Norway	....	....	....	....	....	1	1	1937	110	0
Switzerland	1	1	....	2	2	....	6	1913	148	0
U. S. A.	....	....	1	....	4	2	7	1919	190	0
Yugoslavia	....	1	....	....	....	1	2	1934	120	6
Total	4	10	4	26	20	41	105	....	.....	
Max. span in each group	ft. 121	in. 5	ft. 137	in. 10	ft. 122	in. 6	ft. 232	in. 1	ft. 223	in. 1	ft. 223	in. 4				
Year of completion of max. span	1932		1931		1913		1936		1930		1937					

*Includes maximum span of each type.

standard design was built between 1908 and 1924 over various canals but no development has taken place since the war. The country with the greatest number of large reinforced concrete girder bridges with solid web is Germany where this type is considered to be the most suitable structure for road bridges up to a span well over 100 ft. from the economic as well as from the aesthetic point of view. The rapid increase of the number of such bridges in Japan during the last few years may be specially mentioned.

Among the various types the hinged continuous-beam bridges occupy the dominant position. It is interesting to note that in the groups Nos. 4-6 the maximum spans are almost identical (223 ft.-232 ft.). In groups Nos. 1 and 3 the maximum span attains slightly more than half of this limit (122 ft.). So far as group No. 1 is concerned this fact does not need further explanation. Group No. 3 is the link between beams and arches as a portal with raised centre is to be considered as an arch. In most large span portal bridges it is

possible to raise the centre thus releasing the maximum span moment, consequently most of these bridges are designed as arches.

TABLE 2—CLASSIFICATION OF BRIDGES ACCORDING TO THEIR CROSS SECTION

Cross section	Two or more connected T or L beams	U or H section	Box girder	Solid rectangular slab
No. of bridges....	79	16	9	1

In Table 2 the bridges are subdivided according to their cross section. Here the T-beams are predominant.

TABLE 3—AVERAGE WIDTH PER ONE MAIN GIRDER (98 BRIDGES)

Width in ft. between..	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
No. of bridges.....	1	2	11	11	19	9	10	4	4	7	4	5	5	2	1	.	.	1*					1*	1*

NOTE: In case of U and H sections the width of the main girders has not been taken into account
*H section.

Table 3 indicates the number of bridges according to their average width per main girder. It includes only 98 bridges as the writer could not obtain particulars about the Japanese bridges. Although an average width of about 7 ft. seems to prevail it is to be noted that in modern bridges designers prefer a larger width per main girder if the depth of the girders is not restricted, especially in combination with two-way reinforced slabs. An average width of 15 ft. or even more is very suitable.

TABLE 4— $\frac{D_c}{L}$ = $\frac{\text{DEPTH IN CENTRE OF MAXIMUM SPAN}}{\text{SPAN}}$ RATIO IN 97 BRIDGES

Number of bridges	$\frac{D_c}{L}$ between .015	.020	.025	.030	.035	.040	.045	.050	.055	.060	.065	.070	.075	.080	.085	.090	.095	.100	.105	.110	.115	.120	.125
Type 1								1					1			1						1*	
Type 2				2		3	2	2					1										
Type 3			1	1	1	1																	
Type 4	1	2	4	12			1			4			1						1**				
Type 5				2	1	5	1	4			1	2	1	2	1								
Type 6		1		1		2	3	6	3	4	3	1	5	3	1								
Total.....	1	3	4	16	4	8	9	12	6	8	4	3	6	8	2	1			1			1	

*Railway bridge.
**H section, 23.-10" width per main girder.

Table 4 gives the number of bridges in each group according to the ratio

depth in centre of maximum span : span.

The minimum ratio is .015, a value considered to be very small

even in arches. In usual cases the ratio is within the limits 1/40 to 1/13. Type 1 requires a larger depth than the others. Theoretically the depth in the centre of the span of Types 2-6 can be reduced to any value if the depth at the supports is large enough.

It should be added that by a slight reduction of the limiting span on which Table 1 is based the number of bridges would rapidly increase. Already the inclusion of the limiting span of 100 ft. would bring a considerable number of bridges. The collection cannot of course be complete as there are no doubt bridges within its scope which have not been mentioned in the technical press. The writer will be glad to receive particulars of any other girder bridges which ought to be included.

Discussion, to close in June 1939* JOURNAL, *should reach A. C. I. Secretary in triplicate by Apr. 1.

Vol. 35 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
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Job Problems and Practice

Some will ask questions — Some will answer them — Some will do both

A. C. I. members are invited to use this new JPP department as an informal means toward mutual help. When a problem holds possibilities of general interest the discussion will be briefed in these pages.

If you have a problem—present it; a question, ask it. If you know the answer to a question asked, or if you can contribute something which may help in the solution, or have reason to disagree with the answers or suggestions published, your contribution will be welcome.

If you know of an interesting problem whose solution has already smoothed the way of someone in the field, tell us about it—some other A. C. I. member may need just that information.

How high to back-form arch? (No. 35-10)

Q—To what height will it be necessary to back-form for the placing of concrete in an arch under the following conditions: The arch is a thin slab heavily reinforced requiring thorough vibration of the concrete in order to assure thoroughly sound concrete without any possibility of voids. The design contemplates the use of 6.3 bags of cement per cu. yd. of concrete and maximum of 6.5 gal. of water per bag of cement, which with the aggregates used results in about 4½ in. slump.

A—Under the conditions stated with a well proportioned concrete of 4½ in. slump and thoroughly vibrated it has been found necessary to back-form up to the point where the rise in the top surface of the arch is not more than 6 in. per ft. of horizontal projection. The back forms should have practically continuous lines of removable shutters or windows at a spacing of about 4 ft. vertically. These shutters should be designed for quick installation so that concrete can be placed and vibrated in not more than 4-ft. lifts without interruption. —B. D.

Redesign of large mixers? (No. 35-11)

Q—On a job in New York State where other interests as well as our own were involved in a grade crossing project, truck mixing and delivery were permitted. The mixers were four-yard capacity and inspection of proportions, including water, cement, and time of mix, were rigidly adhered to. I happened to be present when concreting was being done on construction of a section of another interest than our own and was struck with the lack of uniformity of the character of the concrete in different portions of the discharge of the mixer—the first part of the delivery showed segregation and excess of water, the center section showed just about the proper consistency, while the last section was dry and lean looking as if there were not sufficient cement, together with an excess of coarse aggregate in the mix. The remedy resorted to was a longer time of mix and more careful discharge, but it was not a complete cure.

A redesign of the mixers is necessary with a different arrangement of blades and the feed to the discharge, and unless any single project is of sufficient size, a demand of this character will hardly be met. This subject is one for research and study by the ready-mixed industry and the resulting adoption of a type of mixer that will yield a more uniform product when the mixer is discharged. After such improvement is general in the industry, a specification may be written requiring the new type or modification of equipment for use on concrete work where ready-mixed concrete is used.—M. HIRSCHTHAL, Concrete Engineer, D. L. & W. R. Co.

Non-durable chert? (No. 35-7—Nov. '38 Jl. p. 116)

A—The durability of white chalky and vitreous types of chert found in this locality (N. E. Missouri) has been found to depend on the absorption and specific gravity, which are closely related for this material, for as the absorption increases the specific gravity decreases.

By running extensive tests on the specific gravity versus durability in freezing and thawing of the material confined in concrete specimens, it was determined that a chart with specific gravity lower than approximately 2.35 was definitely non-durable.

In order to acquaint inspectors with types of durable and non-durable chert, a solution of potassium iodide in mercuric iodide (Thoulet solution) was diluted to a specific gravity of 2.35 and the light chert floated off. After becoming acquainted with the visual characteristics of the non-durable chert, inspection and acceptance of individual shipments was accomplished by "picking" and weighing of this material.—G. A. A.

Notes from the testing laboratory (No. 35-12)

When concrete cylinders, made in the field in November, show very low strengths no one is surprised. However, when such cylinders, made on a construction job with a nominal 1:2:4 mix and a water-cement ratio of 1.0, showed a strength of 4500 p.s.i. at 7 days, the engineers in charge of a large construction job were both mystified and worried.

On another job, in which the concrete sewer pipe were factory made, strengths as high as 9000 p.s.i. at 28 days were obtained. These cylinders, however, were given 3 days steam curing with the pipe, then stored outdoors or in the laboratory until tested. The concrete in the pipe was vibrated but the cylinders were not.—F. E. R.

Placing concrete against insulation board? (No. 35-13)

Q—A big job for state authorities was to be finished inside the concrete walls with insulation board. The owner's representatives insisted on placing the concrete directly against the insulation board. Contractor objected; owners insisted. The result was sad; the board yielded in places; compressed locally; went to pieces badly. The results are such that it is difficult even to remove the board. The question is, *can* it be done satisfactorily? If so, how?

How effective is hot mixing water? (No. 35-14)

Q—In concreting in cold weather how effective is hot mixing water in bringing concrete to a safe temperature? What is the highest temperature which should be permitted for the mixing water?

A—The effectiveness of hot mixing water as a means of heating concrete depends not only on the temperature at which it is desired to maintain the concrete but also on that of the cement and aggregates from which it is made. If it is assumed that the aggregates are heated sufficiently to eliminate frost therein so that the average temperature of the cement and aggregates is above freezing, then it is possible by the use of hot water alone to raise the temperature of the concrete to above 65° F., assuming that the water used can be heated to about 180° F. If concrete temperatures above 65° F. are wanted then additional heat will have to be supplied to the aggregates.

We believe that the highest practical temperature for the mixing water is around 180° F., and as far as the writer's experience is concerned he has not observed any bad effects from adding water this hot to the concrete.—R. B. Young.

What limits for sodium sulphate test? (No. 35-15)

Q—A County Engineer in Michigan wants to know what limits to apply in the A. S. T. M. sodium sulphate test for soundness to insure durable rock for concrete pavement. The test as described refers only to method. No acceptance limits are stated. The question is, what values are to be applied for normal usage?

A—Current Federal specifications for aggregates (SS-A-281) fix the limit on loss in 5 cycles of the test at 10 per cent for fine aggregates, when the test method is required. However, the test method is not applied when satisfactory evidence is available that the material has proved satisfactory in concrete which has been subjected for at least 5 years to essentially the same conditions of service and exposure as the structure in which the material is to be used.

The Progress Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete recommends "permissible limits" of 8 per cent for fine aggregates and 12 per cent for coarse aggregates; and "maximum permissible limits" of 12 per cent for fine aggregates and 15 per cent for coarse aggregates. These requirements for soundness are waived in the case of aggregates for use in structures or portions of structures not exposed to weathering and, in the case of concrete exposed to weather, when evidence of a satisfactory service record is shown. The report stresses the fact that principal reliance should be placed on service records when adequate information in that regard is available. It also points out that the limits suggested are based on a relatively limited amount of information resulting principally from experience in climates where considerable freezing and thawing is encountered.—STANTON WALKER

Grand Coulee consistency meters (No. 35-16)

For the work at Boulder Dam in 1931, it was felt that something better than visual inspection was needed for determining the consistency of concrete in the mixer before the batch was discharged. A device seemed desirable to indicate the condition of the entire batch, and from which a permanent record could be made. The soluton at that time was the development of the recording watt meter type, which indicated changes in consistency by changes in power required to turn the mixer as the batch varied from dry to wet. Fairly good results were obtained with this type at Boulder Dam. However, this meter applied to a new type of mixer at Grand Coulee Dam, did not function as well as it did at Boulder Dam.

Recently, a new type of consistency meter has been developed at Grand Coulee Dam, and has proved to be much superior to the former type. The new meter is operated by changes in the center of

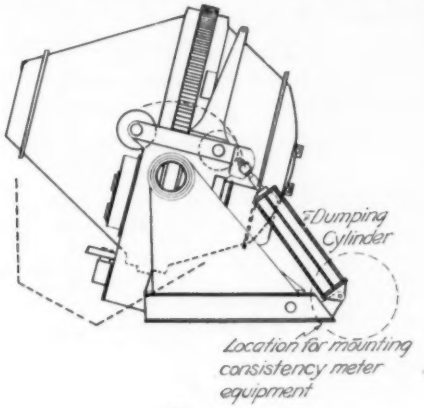

FIG. 1

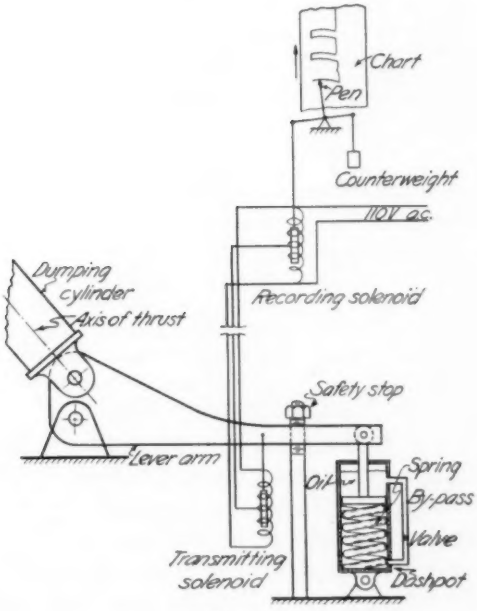

FIG. 2

FIG. 1-2—THE ADAPTATION OF NEW CONSISTENCY METER TO COULEE DAM 4-YD. MIXERS

The spring is enclosed in a small cylinder which is filled with oil and also acts as a dash pot to eliminate vibrations.

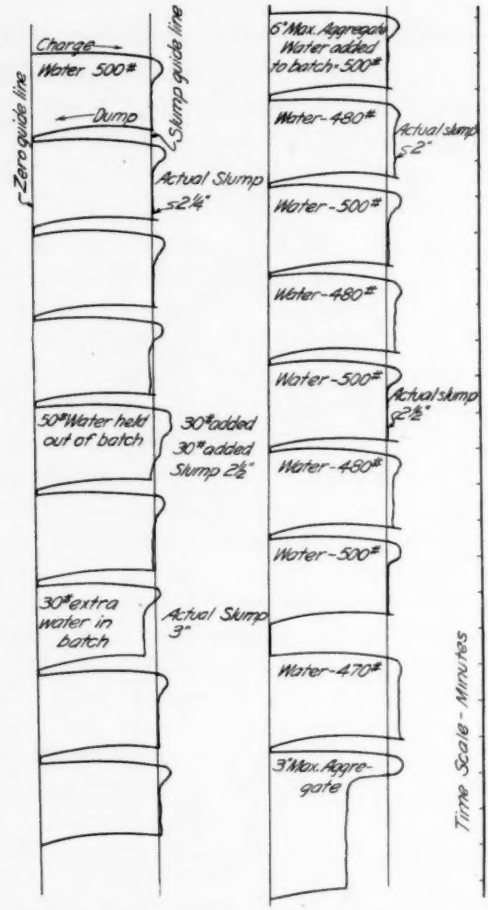

FIG. 3—CHARACTERISTIC RECORD MADE BY TWO METERS

Departure from right-hand guide lines indicates effect of small change in water—a change in slump in inches four times the line variation.

gravity of the batch in the mixer, as the consistency changes. The mixer bowl or frame is supported by springs which deflect with changes in position of the mix in the mixer, as it varies from dry to wet. (Fig. 1, 2) The spring movement is transferred to the indicating and recording mechanism by electrical or other means. (Fig. 3) With this meter, water changes equivalent to less than ½-in. slump are readily apparent. Besides recording consistency changes the device also indicates many other conditions such as erratic batching, changes in mix, starting, stopping, or dumping of the mixer, which is of great value when the mixers are not in plain view. All mixers at Coulee Dam have been equipped with this new type meter.—O. G. PATCH.

Quantities of solids in slurries? (No. 35-17)

Some time ago a friend of the writer's in the building supply business stated that he was making a specialty of preparing and selling an aged lime putty. He said it was made up as a standard putty with a guaranteed lime content, but on storage it lost some water by evaporation, so that a cubic foot contained rather more than the specified amount of lime. He wanted to know if there was any simple way of adjusting the quantity of the stiffer putty to contain the correct amount of lime.

It was suggested that if he would submit a small sample for test, a table or chart could be worked up on the basis of weight per cubic foot that would show at a glance what measured quantity would be equivalent to a cubic foot of the standard putty.

Now Prof. W. C. Voss of M. I. T., in a paper on "The Characteristics of Lime" presented at the annual meeting of the National Lime Association, last May, gives a simple formula to show the content of lime solids in lime putty, which is

$$W_s = 1.7\ (W_p - 62) \quad \ldots\ldots (1)$$

in which W_s and W_p are the weights of solids and putty, respectively. The formula was derived by plotting the weight per cubic foot of a large number of putties of known lime content against the weight of the lime itself, and finding the equation of the line which best fitted the plotted points.

The same type of equation results from the general relation that the weight per cubic foot of any slurry or paste exceeds the weight of 1 cubic foot of water by the weight of the solids minus the weight of the water displaced by them. That is,

$$\text{Excess weight of 1 cu. ft. of paste} = W_p - 62.4 = W_s - \frac{W_s}{g}$$

$$\text{or,}\quad W_s = \frac{g}{g-1}(W_p - 62.4) \quad \ldots\ldots (2)$$

While this formula is strictly applicable only when the solids are insoluble in water, and when the paste contains no entrained air, it is very nearly true for ordinary lime and cement pastes.

In the oil industry, where both drilling muds and cement slurries are commonly used in the construction of wells, it is customary to distinguish between slurries of different "body" by the weight per gallon. The quantity of solids per gallon will, in this case, be indicated by the formula,

$$W_s = \frac{g}{g\text{-}1}\ (W_p - 8.33) \quad \ldots\ldots (3)$$

in which W_p is now the weight of the slurry per gallon. Thus for a 14 lb. cement slurry

$$W_s = \frac{3.15}{2.15}(14 - 8.33) = 8.33 \text{ lb.}$$

The water content is then 14 − 8.33 = 5.67 lb., and the water-cement ratio is 1.5 x 5.67/8.31 = 1.02, or 7.6 gal. per sack. Similar computation for a 16 lb. cement slurry gives, cement 11.27 lb., water 4.73 lb., W/C 0.63, or 4.7 gal. per sack. Thus 14 lb. and 16 lb. cement slurries correspond approximately to pastes of 7½ and 4½ gallons per sack, respectively, and these values are commonly taken as equivalents.

The question might arise, as in the incident first mentioned, how to adjust a paste to a definite content of solids, when the weight test shows it to be heavier or lighter than desired. In this case the weights of water and solids in the standard or desired paste are known, and for the paste in question they are determined from equation (2) or (3), whichever applies. Then the water-solids ratio of the paste to be adjusted must be made the same as that of the standard paste by adding either water or solids in the required amount. The computation involves only simple arithmetic, but if one had to do it more or less frequently, a chart showing the weight of water or solids to be added to a unit volume of paste for any given departure from the standard weight could be drawn from a few computed values. For the dealer selling lime putty, such a chart would most conveniently be a graph showing either weights or volumes of standard putty plotted against weights per cubic foot of putties that would cover the range likely to be encountered.—J. C. P.

How to finish silo interior? (No. 35-18)

Q—In the course of the convention discussion of a paper by Dalton G. Miller, "Factors Which Influence the Durability of Concrete Stave Silos," at the 34th annual convention last February, George P. Dieckmann made rather casual reference to the use of cumar solution, prompting extension of the subject. Referred to Mr. Miller, he wrote:

A—I am satisfied that the rich cement mixes are desirable for mortars to be used on the interiors of silos; that addition of hydrated lime or "mortar mixes" is not necessary and is of questionable merit except insofar as they improve the workability of the mortar from the standpoint of the plasterer. For silo interiors I believe that puzzolanic mortar mixes would be better practice, but I have no data to support this idea. We are trying to perfect a laboratory setup which will make

it possible definitely to determine a number of these uncertain factors in connection with the durability of silos.

Cumar solutions are used extensively by at least two manufacturers in this territory. The practice is to spray the cumar on to the interior cement wash coat or plaster coat immediately after it is applied. Used in this way the cumar aids in sealing in the moisture in the fresh cement wash or plaster coat and improves the curing conditions which at best are always faulty. Cumar hardens to a smooth, varnish like finish and no doubt furnishes some protection against acid action for a time after application, probably a couple of years or so, provided it is not subjected to mechaniacl abrasion when filling and emptying the silo. The cost of a single spray application of cumar for an average silo is not high. One of the users in this territory states it to be from $3.50 to $4.00. Generally one application only is used although two coats are sometimes applied. Sometimes only the lower half of a silo is treated.—DALTON G. MILLER.

The Research Laboratory of the Portland Cement Association carried out a series of tests using various protective treatments and patented materials recommended for treating concrete. These tests showed that Cumar, a relatively inexpensive resin, when used in combination with a suitable oil such as chinawood oil or linseed oil, and thinned sufficiently (naptha, gasoline) so that some of the material would penetrate the concrete, gave a coating of good resistance to silage acids. Cumar dissolved in naphtha or xylol and then mixed with chinawood oil has been used with good success by manufacturers of concrete silo staves. The mixture is sprayed over the fresh inner finish after allowing a few days for surface moisture to evaporate. Most monolithic silos have smooth surfaces which are advantageous in resisting acid. To duplicate this condition stave silos are usually given a trowel or brush coat of cement mortar before applying the acid treatment.—C. L. FORD.

Exterior painting of concrete (No. 35-19)

The Hydro Electric Power Commission of Ontario is the largest producer and distributor of electrical energy in Canada and the owner of a great many properties scattered over the province, which have to be maintained in first class condition. For more than fifteen years the Commission has been studying the problem of painting concrete surfaces. In this time many methods and materials have been experimented with both in the laboratory and the field. From

this experience, certain practices with respect to painting concrete have developed. The compositions of paints are never specified but only brands are used which testing and experience have shown are serviceable for the exposures to which they are to be subjected. Other things being equal, the cheapest paint is used, but quality is never sacrificed to price for the final cost of painting depends more on the number of years during which the job gives protection than on the first cost of the paint.

With this as an introduction, we have been permitted to publish the following letter sent by T. H. Chisholm, Chemist, of the Commission, to A. C. I. Committee 616, Recommended Practice for the Application of Paint to Concrete Surfaces.

In so far as painting concrete is concerned, our experience would indicate that the most important point is to be sure that water does not get into the wall behind the paint film for we have found that such a condition is the cause of the majority of early paint failures on concrete walls.

We have experienced good results on concrete walls with high grade house paints made with a linseed oil vehicle and lead base pigments, provided that proper care is taken in priming the surface. These results have been obtained by thinning the paint with 25 per cent linseed oil for priming and using 10 per cent turpentine or turpentine substitute for the intermediate coat. The chief objection to this practice has been the full oil gloss of the finish. Where the gloss is objectionable, we have used the so-called brick and concrete paints which have a china wood oil vehicle and dry with a satin sheen.

No noticeable advantage has been observed in using china wood oil paints in place of linseed oil paints, and in some cases the maximum life has favored the linseed oil paints. Although china wood oil resists saponification better than linseed oil paints, our observations would indicate that neither material is suitable if moisture enters the wall behind the paint film.

Some tests indicate that the rubber base paints may offer possibilities in painting concrete due to their good alkali resisting properties, but we have had very little experience with such materials as they are not readily obtainable.

Very little work has been done with synthetic finishes on outside walls. On one particular occasion where a bakelite paint was used it gave a very fair life but failed by checking and left a poor surface for refinishing.

Experiments with the so-called cement paints have given very good results in laboratory tests, but on several occasions in the field the results on outside walls have been very disappointing. The engineer in charge of these jobs stated that every detail in application had been closely followed and that the walls were well soaked with water before the materials were applied. In certain locations however, where the surface was subjected to spray or mist, the cement base paints have generally given fair service.

Building code requirements for analysis of fresh concrete? (No. 35-20)

Q—An A. C. I. correspondent in New Zealand, active in building code work, asks whether or not it is common for building codes in the

United States to require the determination of the constituents of fresh concrete.

A—W. M. Dunagan describes "A Method of Determining the Constituents of Fresh Concrete" in the Dec. 1929 A. C. I. JOURNAL, (*Proceedings* Vol. 26, p. 202). (See also Proceedings A. S. T. M. Vol. 31, 1931, Part I, p. 362, "A Study of the Analysis of Fresh Concrete" by W. M. Dunagan; see also p. 383). The question was referred to Prof. R. R. Zipprodt, Columbia University, Secretary A. C. I. Committee 501, Standard Building Code, who writes:

Committee C-9 "Concrete and Concrete Aggregates" of the A. S. T. M. proposed this method for adoption as an A. S. T. M. standard. It does not appear that this action followed.

I know of no building code other than that of New York City (in effect since Jan. 1, 1938) which has included any provision for the determination of the constituents of fresh concrete. Such provision does appear in the new New York code under the head, "Field Tests of Controlled Concrete," as follows:

> During the progress of the work for every 100 cu. yd. of concrete placed of any one strength, a sufficient number of representative samples of the fresh concrete shall be taken as the concrete is being delivered at the point of deposit and segregated into its component parts of water, cement, fine and coarse aggregates. The proportion, water-cement ratio and characteristics of the aggregates shall be determined and compared with the values established by the preliminary tests prescribed in 7.4.3.2.1—*Preliminary Tests of Controlled Concrete.* Any variation shall be corrected immediately by readjustments of the measuring devices, and new proportions and water-cement ratios established if the characteristics of the aggregates have changed materially.
>
> In addition, from the concrete of each strength specified on the plans which is placed on any one day, at least two test cylinders shall be made and stored in accordance with the Standard Method of Making and Storing Compression Test Specimens of Concrete in the Field (D. C. 31-31 of the A. S. T. M.) and tested at the age at which the structure is to receive its full design load, but within 28 days of placing.

The requirement quoted was based directly upon the Dunagan method, although reference to it by name is not made.

—R. R. ZIPPRODT.

How to design concrete mixtures? (No. 35-21)

Inquiries about the design of concrete mixtures, almost always looking in the direction of simplification, are numerous. They arise usually out of what seem to be special conditions where convenient short cuts are needed. With no intention to prejudice the current labors of A. C. I. Committee 613—Recommended Practice for the

Design of Concrete Mixes, R. F. Blanks, Chairman, it seems proper to discuss the problem in these pages whenever new or additional light can be thrown on the subject, or when a problem of mix design can be approached from a new or special angle which may be helpful to *some* A. C. I. members.

As a starter we append excerpts from a recent letter by Stanton Walker, Director of Engineering, to members of the National Sand & Gravel Association, as a result of the frequent requests for recommendations as to the proportions of materials required to produce concretes of different strengths. Other members of the Institute are invited to participate in further suggestions along lines which may prove to be helpful.—EDITOR

While many methods for the design of concrete mixtures have been advanced, most of them having a great deal of merit, none of them is sufficiently accurate to arrive at "guaranteed" strengths unless supplemented by laboratory tests. They are of principal usefulness, and this is an important function, in arriving at the most economical proportions—i. e., those which will give the highest strength for a given cement content.

The chief difficulty in designing for a specific strength, without the aid of laboratory strength tests, comes from the wide range in the strength-producing properties of different cements. Further, of course, no design for strength should ever be made which contemplates "guaranteeing" the strength of field cured specimens. All such designs should be based on laboratory curing. Because of differences in temperature and curing conditions in the field, there is no necessary direct relation (particularly at early ages) between the strength of concrete in the job and that cured under laboratory conditions. However, strength tests of specimens cured under standard conditions (moist and at a temperature of 65° to 75° F.) do provide an accurate criterion of the potential strength of job concrete.

A fault of most design methods which have been advanced is that they are either too technical or too detailed to be attractive to the average practical engineer who does not specialize in the design of concrete mixtures. Nevertheless, such engineers do feel the need, from time to time, for a simple procedure which will furnish an approximate idea of the strength which will be obtained for given proportions, or the proportions required to produce given strengths.

In the following there is described a simple procedure. There is nothing new about it. It is based on well-understood principles and is used by a number of engineers. It does not take into account differ-

ences in grading of aggregate, except as to maximum size, but it does fit a wide range of aggregates with surprising accuracy.

Step 1. Assume, or otherwise arrive at, a water-ratio strength relationship. Table 1 gives a relatively "conservative" one based on average moderate-strength cement and a "probable" one based on results frequently obtained with modern cements.

TABLE 1

Compressive Strength at 28 Days p.s.i. (Standard Curing)	Water, Gallons per Sack of Cement	
	Conservative	Probable
2000	8	8½
2500	7½	8
3000	6½	7¼
3500	5¾	6½
4000	5¼	6

Step 2. Assume, or otherwise arrive at, the quantity of mixing water per *cubic yard of concrete* required to produce the desired plasticity with the aggregate in question. For all practical purposes of this fairly "rough-and-ready" method of design, the quantity of mixing water per *cubic yard of concrete* will be the same for different strengths—if the aggregate and the plasticity (slump) are the same. "Order of magnitude" values for average well-graded sand and gravel aggregates are given in Table 2:

TABLE 2

Maximum Size of Gravel	Gallons of Water per Cubic Yard of Concrete (Assumed Same for All Strengths)		
	1-2 In. Slump	3-4 In. Slump	6-7 In. Slump
¾"	35	38	42
1"	33	36	40
1½"	32	35	39
2"	30	33	36

Step 3. Determine the number of sacks of cement per cubic yard of concrete. That can be done directly from the data in Tables 1 and 2. Merely divide the number of gallons per cubic yard of concrete by the number of gallons per sack of cement. For example, under these assumptions, and using the "conservative" column for gallons per sack, a 3000 p.s.i. concrete having a 3 to 4-in. slump would be produced with a well-graded 1½-in. top-size aggregate using 35 gal. of water per cu. yd. of concrete (total, of course, including moisture) and $\frac{35}{6\frac{1}{2}} = 5.4$ sacks of cement per cu. yd. of concrete. Similarly, a 2000-p.s.i. concrete with a 3 to 4-in. slump would be produced with the 1½-in. aggregate using, *again* 35 gal. of water per cu. yd. of concrete and $\frac{35}{8} = 4.37$ sacks of cement per cubic yard of concrete. Simple, isn't it? and, just about as accurate as more complicated methods if you know your aggregates to be of reasonably normal grading.

Step 4. Determine the quantities of total aggregates per cubic yard of concrete. There are two simple methods. The first (A) is the easier and the less accurate. The second (B) is not much harder and much more accurate.

A. Assume a weight per cubic yard for your concrete. If you don't know it, consider 4000 lb. close enough for normal gravel-aggregate concrete. The weight

may be a little higher for your coarser gradings and a little lighter for your finer gradings but that figure is a pretty good average. Now, you know the weights of cement and water; just subtract them from 4000 lb. and you have your total aggregate. For example, in the first example given in Step 3, we have

35 gallons of water = 292 lb.
5.4 sacks of cement = 508 lb.
Total = 800 lb.
Sand and gravel = 4000-800 = 3200 lb.

B. The more accurate method requires a knowledge of, or an assumption for, the specific gravity of your aggregates. Assume, for the purpose of this illustration, that the specific gravity is 2.63—and that is not a bad general value to assume. Then a *solid* cubic foot of your material (visualize one big piece of gravel a cubic foot in volume) weighs 2.63 x 62.5 = 164 lb. A further assumption is that each 94 lb. sack of cement contains ½ cu. ft. of solid cement (.0485 is the more generally used figure).

From the preceding we can calculate the volume of the cement paste (it is the volume of solid cement plus the volume of mixing water) per cubic yard of concrete. That quantity subtracted from 27 cu. ft. gives the number of cubic feet of *solid* aggregate and that number multiplied by 164 gives the pounds of total aggregate.

For example, in the first example given in Step 3 (the same as used in B) we have

35 gal. of water = 4.66 cu. ft.
5.4 sacks of cement = 2.70 solid cu. ft.
Total = 7.36 cu. ft. of paste
27 — 7.36 = 19.64 cu. ft. of solid aggregate
19.64 x 164 = 3220 lb. total aggregate

Step 5. Determine the proportion of sand and gravel in the total aggregate. That depends to such a large extent on the grading of the aggregates that it is difficult to give approximations. However, you know fairly well what the proportion should be from past experience and, in any event, it should represent the least quantity of sand which will produce a smooth workable mix. Some figures to shoot at are given in Table 3.

TABLE 3

Maximum Size of Gravel	Sand Percent by Weight of Total Aggregate	
	Lean Mixes	Rich Mixes
¾″	50	40
1″	47	37
1½″	43	33
2″	40	30

Step 6. Having completed all of your calculations it will be a good idea to boost all quantities about 1 or 2 per cent, as a factor of safety.

This simple method has been represented as approximate. However, it is only as approximate as your assumptions. If you know the water-ratio strength relationship applicable to your materials and to the cement to be used and if you know, from experience, or trial, the gallons of water per *cubic yard of concrete* required to produce the desired workability, accurate results will be arrived at.—STANTON WALKER.

Coloring concrete by absorption? (No. 35-22)

Q—From California comes a request for information on coloring concrete by absorption, using solutions of metallic salts.

A—More than 25 years ago Adolph Schilling described his use of the absorption method in coloring concrete as follows:*

The absorptive qualities of concrete during its state of curing and seasoning offer opportunities for coloring concrete products by capillary action. By this method, the color is deposited in the pores of the surface, amalgamating with the concrete in a permanent unit. The possibilities of this treatment are unlimited, but individual knowledge of coloring values and judgment, so as not to impair the strength requirements of concrete, are essential to success.

Coloring solutions can be made to penetrate the surface of concrete 6 in. or more, if the object is placed in the solution in a very green state. It is rarely necessary to penetrate more than $\frac{1}{32}$ in. to $\frac{1}{8}$ in.; this thoroughly fills all pores, gives the desired color effects, and is less expensive.

Every atom of coloring matter absorbed by the concrete reduces the strength of the solution; and as some of the coloring matter used is expensive, good judgment in allowing only the necessary absorption of coloring matter is advisable from an economic standpoint.

Aniline colors and the sulphates of copper (1 to 2 lb. to 3 gal. of water) and iron (1 to 2 lb. to 2 gal. of water) are the most suitable to make solutions in which to color concrete by the capillary method.

The concrete to be colored can be treated after it is several days old. Concrete products with strength requirement should not be subjected to the coloring bath until the concrete has attained its required strength, as the filling of the pores in the concrete stops the action of its curing by the usual methods.

Coloring by absorption is effective on surfaces of concrete after it comes out of the mold, or after being treated with acid or tools. Surfaces that have been colored by absorbing mineral or metallic colors become waterproof, and the action of the weather on the metallic colors is the same as on real metals, increasing the beauty of coloring by the usual oxidation noticed on bronze and copper. Surfaces of concrete treated by this method become so hard and dense that they will take a uniform dull or high gloss polish. I have treated such surfaces in the same manner as marble, granite and metal, under polishing or buffing machines.

Products made by these methods, such as flower pots, vases and boxes, will hold water after the second day of casting, and become so hard that when struck with a hammer they ring like a metal bell. I do not think that waterproofing compounds are essential in obtaining this result, but consider the proper amount of water and thorough grading of the aggregate as all important.

Ordinary concrete can be made very attractive by one coat or two coats of waterproof paint and if applied in stipple fashion it will not impair the grain or texture, avoiding the undesirable effect of painted stone.

This method is especially to be recommended for dry or semi-dry process concrete work, as the porous surface readily absorbs the waterproof liquid, and allows the pigment particles to fill the pores.

The color effect obtained in this manner is the most economical, but it is a flat color, and does not give the richness and depth of shading that results from the

*"Concrete" Jan. 1913, p. 6. Further reference: May, 1914, p. 216 and Feb. 1917 p. 60.

immersion treatment in an aniline or metallic color bath. It has the advantage, however, of being used in cases in which immersion is not practicable.

I have obtained excellent two- and three-color effects by painting certain parts of objects before subjecting them to the coloring bath. The parts so colored would not be affected by the color in the bath.

The artistic possibilities of such treatment are limited only by the color sense and taste employed by the craftsman. By using certain non-absorptive aggregates, their natural color can be retained, while the absorptive parts will assume the desired color. In this treatment, precaution must be taken not to use certain acids in washing before immersion in the color bath, as the chemical action of the acids is likely to counteract the color value of the bath.

Long practice and tests will give the experience necessary to gain the full benefit of this process.

Curing test cylinders in the field? (No. 35-23)

Q—Can you suggest a simple but uniform method for curing concrete test cylinders in the field?

A—An unique and effective control method for curing concrete test cylinders may be accomplished by using a small portable galvanized tank large enough to inundate about 3 dozen 6 x 12 in. cylinders. This tank is made of two sheets of galvanized iron with an electric immersion heater at each end. The temperature is controlled by an immersion type thermostat. The total cost of this equipment is about $25.00.—WILFRID SCHNARR.

While this JPP department is not for treatises, it may provide fragments which next year's treatises will note. It will provide a place where ***A***, who is in a jam, meets ***B***, who was once in a similar jam and got out of it, or who knows somebody who was and did—and how.

The "answers" are the answers of individuals to whom the questions are referred and not of the A. C. I. as an organization.—EDITOR

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Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Specifications (German) for the construction and testing of concrete pipe

Zement 27(39), Sept. 29, 1938, pp. 611-618; 27(40), Oct. 6, 1938, pp. 634-635.
Reviewed by L. T. BROWNMILLER

The specifications given are the tentative specifications for concrete pipe.

The (static) effect of the superstructure on large arch bridges

ULRICH FISCHER, *Beton und Eisen*, Vol. 37, No. 19, p. 310, Oct. 5, 1938 Reviewed by A. U. THEUER

The author discusses the effect of superstructure on arch ribs and slabs. Observations made on completed arches provide evidence for the author's conclusion that the effect of the superstructure almost invariably tends to decrease the stresses in the arches themselves.

Determination of free lime

J. W. DENNIS, *Rock Products*, Dec. 1938, Vol. 41, No. 12, p. 43. Reviewed by ROY N. YOUNG

A rapid method for the determination of free lime in portland cement requiring only 5 to 10 minutes has been developed. The procedure is similar to that recommended by Lerch and Bogue except (1) a 2 to 1 alcohol-glycerol solution is used instead of a 5 to 1 solution; (2) 2 grams of anhydrous strontium nitrate is employed as an accelerator, and (3) titrations are made at 5-minute intervals.

Concrete road construction in Germany

Fachgruppe Zement-Industrie, Berlin—Charlottenburg, pp. 1-82, 1938. Reviewed by F. L. EHASZ

Statistical and technical progress in the construction of concrete roads in Germany over the period 1925-1937 is reported in this publication, with most emphasis on the last four years. Sections are devoted to the recent specifications, development of cement, machines and devices in general. Others give a survey of the various construction methods and a tabular record of concrete road construction in other countries as compared with German statistics. The booklet is profusely illustrated and serves well in setting forth recent developments by German engineers.

Aerated barley silos of the Bamberg Malt Co.

A. GUT, *Beton und Eisen*, Vol. 37, No. 19, p. 305, October 5, 1938

Following a general description of the layout of a large malt factory with its system of grain elevators, the author gives a detailed description of the aerating system of silo construction as devised by Dr. J. Rank of Munich. In this system the silo walls

consist of hollow masonry units so placed that their hollow spaces provide the necessary air canals. Vents are placed directly underneath the overhang of horizontal concrete framing bents. The system of aeration is reversible and can be limited so as to be operative up to a given height of elevator only.

Development and present position of photoelasticity and photoplasticity

R. V. BAUD, Special reprint from *Schweizer Archiv.* (Nos. 1 and 2, 1938), *EMPA*, Zurich, Report No. 118, pp. 1-23. Reviewed by F. L. EHASZ

Recent development of analogy methods for determining the sum of principal stresses at interior points of a body subjected to two-dimensional stress and the extension of photoelastic studies to three-dimensional problems are the high lights of this compendious report, compiled by a veteran in photoelasticity. An account of the fundamentals and a systematic tabulation of the various methods employed in this field are presented. Attention is called particularly to the author's analogy of the "electrolytic trough" method.

Homes and Gardens

Homes and Gardens Number, *Indian Concrete Journal*, Vol 12, No. 9, Sept., 1938.
Reviewed by J. C. PEARSON

Those who have access to this Journal from far away India, and who enjoy unusual photographs of palatial homes and gardens, will be interested in the extent to which concrete is used for architectural and decorative effects in that country. Almost the entire space of this September issue is given over to views illustrating the adaptation of concrete to residences, garden ornamentation, swimming pools, etc. One group of photographs was taken on the estate of a Nawab of Hyderabad showing a highly modernistic and yet exotic palace with gates, porches and terraces, all in concrete. The group bears the legend, "These pictures were not sent us from Hollywood!"

The determination of admixtures in cement

K. KEIL and F. GILLE. *Zement*, Vol., 27 No. 40 and 41. Oct. 6 and 13, 1938, pp. 623-629, 640-643.
Reviewed by L. T. BROWNMILLER

There is nothing particularly new in this article but it gives, in comprehensive form, some of the important properties of common cement admixtures which may be used for their identification. Such admixtures include chlorides, coloring materials, water glass, organic materials, raw materials, slags, puzzolanas, etc. For qualitative identification, microscopic analysis is recommended. For quantitative measurements of the extent of admixture, planimetric analysis and a separation of the admixture on the basis of specific gravity are recommended with subsequent chemical analysis of the fractions.

Swedish built bridges in Iran

SVEN KLINGENBERG, *Teknisk Tidskrift*, Vol. 68, No. 34, Aug. 27, 1938, pp. 391-401.
Reviewed by INGE LYSE

In recent years a revolutionary construction development has taken place within the borders of Iran, carried out to a considerable extent by Scandinavian engineers and contractors and the article gives an interesting description of extensive bridge construction, done by a Swedish concern. In the relatively short period of the last 4 years this firm has constructed 60 bridges of span lengths varying from 36 to 1640 ft. Four of them were railroad bridges, stone arches, five were street bridges and the rest highway bridges. The great majority of these bridges were constructed of reinforced concrete. Because of poor foundations most of the bridges were statically determined and a considerable number of three hinged reinforced concrete arches were used. The article contains 27 illustrations and photographs.

From the practice of sliding formwork

FRANZ BOEHM, *Beton und Eisen*, Vol. 37, No. 19, October 5, 1938. Reviewed by A. U. THEUER

The adaptability of hose pipe-scales for use with the sliding form method of construction leads the author into a discussion of the characteristics of this type of weighing device. Precautions to be observed in use are listed. The problem of rising vertically and keeping walls plumb is then taken up. In considerable detail he discusses the special case of unit construction involving vertical expansion joints between successive units. Appearances here require a plane surface across the expansion joint and a rise parallel to that of the surface of the completed unit, both problems requiring special precautions on the job. In this connection the bending or bulging) of forms due to the lateral pressure of freshly poured concrete is brought out.

Experiments for the development of an accelerated test for shrinkage

G. MUSZGNUG. *Zement* 27, No. 39, Sept. 29, 1938, pp. 605-611. Reviewed by L.T. BROWNMILLER

Cements for the German Reich auto highways must not have a shrinkage of more than 0.55 mm. per m. at 28 days in general. Accordingly the author has attempted to develop an accelerated test to predict the shrinkage at 28 days.

Most of the experiments were conducted with two types of cements, a portland and an iron portland cement. The author's approach to the problem was to determine the water given off at various ages and under different conditions of humidity and vacuum. The shrinkage of the cements under all conditions was measured. Detailed results are given in a series of curves. The conclusion is that none of the experiments in the series gives reliable results for predicting the shrinkage at 28 days by an accelerated test. The data may be valuable to someone attempting to develop such a test.

Tests of reinforced concrete hinges of the Mesnager type

G. C. ERNST, Highway Research Board, 1937. AUTHOR'S SYNOPSIS

Sixty-two reinforced concrete hinges of the Mesnager type were tested as the beginning of an investigation at the University of Maryland upon the principal hinge types used at present in rigid frame construction. Bar sizes of ¼, ⅜, and ½ in. were used with various combinations of rotation, thrust, and shear. The bars, with $1/r$ values of approximately 40, 65, and 90, were crossed at 15, 30, and 45 deg. with the center-line of the hinge.

It was found that for these hinges, (1) proper placing and wiring of the transverse steel was essential, (2) the ultimate load was practically a linear function of the number of bars, (3) rotation of the top block affected the ultimate thrust only slightly for rotations encountered in practice, (4) increase in load capacity could be obtained by the use of high yield point steels, (5) the ultimate load was reduced with increases in the $1/r$ ratio of the bars, (6) computed stresses indicated a satisfactory agreement between tests and the equations presented by Parsons and Stang in *Proceedings*, A. C. I. Vol. 31, p. 304. (7) combinations of thrust, shear, and rotation were not particularly severe unless the rotation was clockwise with shear to the right (or counter-clockwise with shear to the left.)

Determination of two-dimensional potential fields for arbitrary boundary conditions

R. V. BAUD and F. TANK, Special reprint from *Schweiz. Bauzeitung*, v. 111, No. 14, April 2, 1938. Reviewed by F. L. EHASZ

This brief paper brings to the readers' attention a new method of obtaining the sum of principal stresses at interior points in two-dimensional problems. The analogy between the Laplacian equation for invariant sum $(\delta_x + \delta_y) = (\delta_1 + \delta_2)$ of normal

stresses and for the potential function of the electrostatic field is resorted to in solving two problems of elasticity. An electrolytic trough, a potentiometer and a voltmeter constitute in large part the equipment for measuring and comparing potentials at various points of the model which consisted of eternite. A boundary in conformity with results of any of the methods for evaluating the difference in principal stresses was set up with due regard to precautions involving electrical conduction and heat formation. Difficulties in the earlier tests were mitigated, once a satisfactory mixture of ising glass and graphite with the required resistance was found. At various chosen boundary points the required potentials were supplied and the necessary number of measurements in the interior made. Experiments for the problems of linear loading of the half plane and the circular hole in a uniaxial field of stress gave results which compared favorably with the theory of elasticity. Desire for improvement in the method of measurement was expressed by the authors.

Earth pressure in road and bridge construction

LEO RENDULIC, *Forschungsarbeiten aus dem Strassenwesen*, V. 10, pp. 1-84, 1938.
Reviewed by F. L. EHASZ

Soil mechanics in Germany has been given much consideration in connection with important bridge and building projects in recent times. Need for a recapitulative account of old and new developments in this field was realized and satisfied in large measure. Many periodicals in several languages were reviewed for the preparation of this paper. The reader is left to decide for himself on controversial and contradictory statements which have come to light in the literature and ascertain which aspects are valid for his particular problem. It is pointed out that the implied assumptions in the classical earth pressure theory must be kept clearly in mind and the theory applied only in cases where these conditions are fulfilled. The classical theory may be modified in the author's opinion for unyielding walls and a high center of rotation, but by no means replaced by a completely new theory. A new school of thought would discard the classical methods and by means of the theory of elasticity reshape the methods from fundamentals. In the present paper, however, the Engesser geometric earth pressure theory is considered very highly, the author believing it to be advantageous because of its lucidity and simplicity. Investigation of pressure distribution on the base of a dam is illustrated with the aid of Engesser-Hull curves. The topics treated are summarized as follows: (1) earth pressure and resistance in the case of sand; (2) distribution of pressure along retaining wall; (3) earth pressure on unyielding retaining walls; (4) influence of time on supporting constructions; (5) practical determination of earth pressure; (6) earth pressure under water; (7) distribution of pressure of a dam on its foundation; (8) Mohr stress representation; (9) earth pressure on wing walls normal to axis of dam; (10) allowable earth pressure in the case of retaining walls and abutments; and (11) earth pressure in cohesive soils.

Formation of hydrated calcium silicates at elevated temperatures and pressures

EINAR P. FLINT, HOWARD F. MCMURDIE and LANSING S. WELLS, *Research Paper* RP 1147—National Bureau of Standards, Vol. 21, No. 5, Nov. 1938 AUTHOR'S ABSTRACT

An X-ray investigation of the crystalline hydrated calcium silicates which occur naturally as minerals indicated that the following are distinct compounds: Okenite, $CaO.2SiO_2.2H_2O$; gyrolite, $2CaO.3SiO_2.2H_2O$; crestmoreite, $2CaO.2SiO_2.3H_2O$; xonotlite, $5CaO.5SiO_2.H_2O$; afwillite, $3CaO.2SiO_2.3H_2O$; foshagite, $5CaO.3SiO_2.3H_2O$; and hillebrandite, $2CaO.SiO_2.H_2O$. Riversideite is apparently not a distinct mineral and is probably the same compound as crestmoreite. Crystalline preparations having X-ray patterns identical with those of gyrolite, xonotlite, and foshagite

were synthesized by hydrothermal treatment of calcium silicate in the form of the anhydrous crystalline compounds, glass, and amorphous hydrate. Other compounds formed were cristobalite, wollastonite, pseudowollastonite, $4CaO.5SiO_2.5H_2O$, $CaO.SiO_2.H_2O$, $10CaO.5SiO_2.6H_2O$, $2CaO.SiO_2.H_2O$, and $3CaO.SiO_2.2H_2O$. A low-temperature modification of anhydrous tricalcium disilicate, which inverts to the usual form at 1,024° ± 5°C, resulted from the action of water vapor on tricalcium disilicate at 500°C, 380 atmospheres. Gyrolite was obtained at temperatures between 150° and 400° C, $4CaO.5SiO_2.5H_2O$ at 150° to 275° C, $CaO.SiO_2.H_2O$ at 150° C, xonotlite at 175° to 390° C, wollastonite at 400° C. and higher, foshagite at 300° to 350° C, $10CaO.5SiO_2.6H_2O$ at 100° to 200° C, and $3CaO.SiO_2.2H_2O$ at 200° to 450° C. Attempts to prepare okenite, crestmoreite, afwillite, and hillebrandite were unsuccessful. Treatment of tricalcium and dicalcium silicates with water at room temperature for several years gave crystalline alteration products which were different from other known hydrated calcium silicates. That derived from dicalcium silicate had a composition approaching $3CaO.2SiO_2.1.5H_2O$. Attempts to isolate and identify the hydrated calcium silicate binding material in commercial sand-lime brick and in laboratory-prepared brick were unsuccessful, but a partial separation of the cementing material was apparently obtained. Applications of the results to the autoclave testing of cements are discussed.

Notes on rendering (stucco)

ANNONYMOUS. *Concrete Building & Concrete Products.* Vol. 13, No. 10, Oct. 1938, p. 206.
Reviewed by J. C. PEARSON

These notes, from the Report of the Building Research Station for 1937, discuss certain principles of rendering (stucco to us) that should be known to plasterers and specification writers. A common trouble with rendered finishes is the development of shrinkage cracks. Rain water is drawn into these by capillarity, and if the rendering is dense, external evaporation is slight and the wall is kept damp. Thus a wall covered with a dense, but cracked, plaster may actually be less weatherproof than the bare wall would be. Remedying such a condition is almost impossible.

Another defect is irregularity in color which results from different drying rates. This is characteristic of wood-float finishes, and the blemishes are due to the differences in porosity and texture. Such finishes soon look shabby for uneven discoloration. This condition has been largely eliminated by the practice of scraping off the surface skin of the stucco a few hours after application.

A strong, impermeable cement-sand stucco is regarded as less suitable for weather protection than a weaker mix, because of the more conspicuous and more open cracks in the former, and the more ready evaporation of moisture from the latter.

Protection of projections and horizontal surfaces by zinc or copper flashing is always recommended, because any feature which allows water to penetrate the masonry behind a rendering is liable to cause harm.

It is the prevailing practice on the continent to apply renderings by dashing rather than trowelling. This is believed to make for good adhesion, uniformity of texture, and reduction of crazing. Rough textures are preferred, not only for uniformity of appearance but for greater weatherproofness.

The usual rendering material is a mixture of lime and cement in which the latter is one third or less by volume of the lime, and the combined volume of cement and lime is one third that of the sand. A mix which has given good results in experimental work is 1 part cement, 3 parts lime and 10 to 12 parts sand. The objective seems to be that which is slowly coming to be realized in the United States, namely, that shrinkage is a much more potent enemy of good stucco than porosity.

Water permeability of masonry walls

CYRUS C. FISHBURN, DAVID WATSTEIN, and DOUGLAS E. PARSONS, Report, Building Materials and Structures, 7, Oct. 18, 1938. AUTHORS' ABSTRACT

Walls of brick, structural clay tile and hollow concrete unit masonry were tested under conditions resembling exposures to wind and rain and their resistance to the penetration of water determined. Five kinds of workmanship (method of filling joints), three kinds of brick, six kinds of structural clay tile, two kinds of hollow concrete units, and six different mortars were used in the construction of the walls. One group of 48 brick walls included specimens, both 8 and 12 inches thick, of three kinds of brick, four cement-lime mortars, and two classes of workmanship, in all possible combinations. The walls of structural clay tile were faced either with stucco or with brick of a single variety. Those of concrete units were faced either with the same brick or with cement paint. The effectiveness of several methods of "waterproofing" were compared by retesting some of the walls after they had been treated. The treatments for the walls included the filling of openings in the face joints with mortar, grout, or wax, and the application to the exposed surfaces of colorless solutions or paints. Most of the walls were subjected to water penetration tests under two different conditions. Water was applied near the tops of the walls in a manner to produce a thin film of water over their exposed surfaces. In the "capillarity" test no pressure was applied to the exposed face, the water penetrating under the forces of capillarity and gravity only. In the "heavy-rain" test a static air pressure of 10 pounds per square foot was maintained against the exposed face.

The performances of the walls in the water penetration tests depended more upon the quality of the workmanship than upon any other factor. Walls of brick having the interior joints well filled with mortar usually gave excellent performances, whereas those with poorly filled joints leaked. Aids in obtaining walls resistant to moisture penetration were the use of mortars of medium or high water retentivity, the wetting of absorptive brick before use, and the application of a parging of mortar on the back of the facing wythe. The omission of two-thirds of the normal number of header bricks, or the insertion of a limestone sill or belt course had no important effect on the permeability of the walls. On the average, walls with a brick facing and a backing of hollow masonry units were slightly less permeable than brick walls of equal thickness when the joints were not well filled. When the joints were well filled the performance of walls with hollow units was somewhat superior in the capillarity test but inferior in the heavy-rain test to that for otherwise similar all-brick walls. The performances of walls of structural clay tile faced with portland-cement stucco was somewhat better than the average for the walls of brick. The filling of openings in the joints with mortar, grout, or wax was effective in stopping leakage. Applications of colorless waterproofing solutions did not stop leakage through openings in the joints, but were effective in improving the performance of walls of absorptive brick when the openings in the joints had been sealed. Coatings of molten paraffin, oil paint, and cement paint were effective in reducing moisture penetration.

Eccentrically-loaded reinforced concrete columns

CHARLES S. WHITNEY, *Concrete and Constructional Engineering* (England) V. 33, No. 11, Nov. 1938, p. 549-561 Reviewed by the AUTHOR

A simple, rational method for the design of eccentrically loaded columns is presented. New formulas based on the author's previously published equations (Journal A. C. I., V. 33, p. 483) put in more convenient form and improved by analysis of recent tests published by F. G. Thomas and Messrs. Richart and Olson. They appear to be in good agreement with about 180 tests covering a wide range of conditions.

They are based on the cylinder strength of the concrete and the yield point strength of the steel without involving the modular ratio, and apply to both round and square cored columns with small or large eccentricities.

In their recent work, Richart and Olson suggest that design should be based on the transformed section modulus of the uncracked section, involving the modular ratio which is variable and uncertain. For the allowable unit stress they propose an empirically varying value from 0.225 f'_c for a concentric load and approaching 0.4 f'_c for larger eccentricities. This method is obviously largely empirical and laborious and could only be justified because of agreement with tests. No solution is offered for the case where the strength is limited by the tensile strength of the steel, which is probably more common than the case of very small eccentricity where the compressive strength controls.

The method suggested by Doctor Thomas, using the present formulas based on the straight-line no-tension theory, covers the full range of eccentricities but is even more difficult to apply to round columns than the Richart and Olson method. Thomas also introduces a variable allowable concrete stress in the range between zero eccentricity and $e = 0.5d$.

The formula derived by the author for column load as controlled by the compressive strength of the concrete is represented by a rectangular hyperbola which makes the adjustment between column and beam strengths without the auxiliary empirical curves of the other two methods.

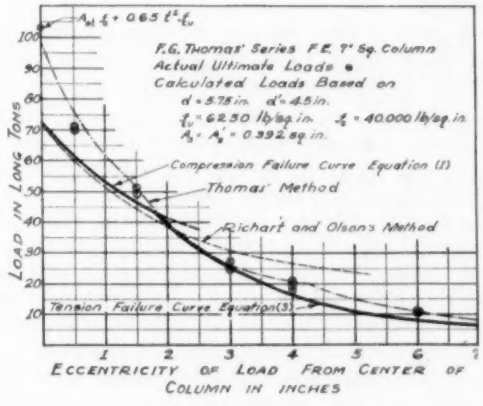
$A_{st} f_s + 0.65 f'_c f_u$
F.E. Thomas' Series F.E. 7" Sq. Column
Actual Ultimate Loads •
Calculated Loads Based on
d = 5.75 in. d' = 4.5 in.
f_u = 62.50 lb/sq. in. f_s = 40,000 lb/sq. in.
$A_s = A'_s$ = 0.392 sq. in.
Compression Failure Curve Equation (1)
Thomas' Method
Richart and Olson's Method
Tension Failure Curve Equation (3)
LOAD IN LONG TONS
ECCENTRICITY OF LOAD FROM CENTER OF COLUMN IN INCHES

FIG. 1

For rectangular columns with ties the formula proposed for allowable load for the smaller eccentricities when the failure would be by compression is as follows:

$$P = \frac{0.64\, A'_s f'_s}{\frac{2e}{d'} + 1} + \frac{0.272\, A_c f'_c}{\frac{2.55te}{d^2} + 1.51}$$

This reduces to the standard formula for axial loading.

When the eccentricity is greater and failure would be by tension in the steel, the allowable load would be given by the following formula if a factor of safety of two and one half is desired:

$$P = 0.4tbf_c\left[\sqrt{\left(\frac{e}{t} - 0.4\right)^2 + d'p_tm} - \left(\frac{e}{t} - 0.4\right)\right]$$

in which, p_t is the total percentage of longitudinal steel and m is the yield point strength of the steel divided by 0.85 times the cylinder strength of the concrete.

Formulas are also given for unbalanced reinforcement and for round or square columns with round cores. Loads calculated by these formulas are compared with test results. The author believes that these formulas provide a simple and consistent treatment for the full range of members from the axially-loaded columns to beams under flexure without direct load.

Concrete manual

A manual for the control of concrete construction: Issued by the Office of the Chief Engineer, Bureau of Reclamation; Prepared and edited under the direction of A. RUETTGERS and R. F. BLANKS, U. S. Bureau of Reclamation, Denver, Colo. and Washington, D. C. ($1.00)

Reviewed by G. W. HUTCHINSON

The "Concrete Manual", in general, recognizes practically all the details admitted as important in the present stage of concrete knowledge. Although intended for guidance of the field forces of the Bureau of Reclamation, it serves admirably as a text book covering the various subjects incidental to the design and control necessary for modern concrete. Using the water-cement ratio theory as a basis, numerous charts and curves, derived from the results of investigation, indicate the effect of cement, grading of aggregate, curing, temperature, size of test specimen, etc., on the properties of concrete. It illustrates, graphically, the factors which contribute to the production of good, uniform concrete and discusses them in their relation to durability, strength and economy. The investigations are not limited to the work of the Bureau alone, but include researches by other recognized authorities.

Desired properties of concrete with suitable materials are to be most economically obtained through the use of the lowest practical water-cement ratio; placement by mechanical vibration; and adequate curing. Emphasis is placed on the curing of field concrete and photographic evidence indicating the effect of different curing conditions is offered. With reference to proportioning, data are presented in a manner to allow practical application to any type of concrete construction. They are supplemented by tables which direct the procedure for trial mixtures. Correction factors and allowances for making further adjustment are included. The compilation of mix tables are based upon the absolute volume relationship; practical experience regarding the relation of water content to consistency; and data in connection with the fine-coarse aggregate ratio secured from tests of natural sand and gravel. Several typical portland cements and their characteristics are discussed. The differences in compound composition, strength, heat of hydration, and the temperature rise of concrete containing them, are indicated. Various chemical solutions and the degree of their effect on unprotected concrete are listed. Improvement in control of the leaching of soluble compounds by water percolating through the concrete, or through cracks and joints, is pointed out through the use of special cements which liberate less free lime upon hydration, but such faults may be minimized by the production of uniformly watertight concrete with relatively low water ratios; vibration compaction; adequate curing; and proper treatment of the concrete surfaces at construction joints. Various methods in use for the protection of concrete placed in hot and cold weather are described. It touches on the proper treatment of concrete surfaces as well as upon foundations which are to support the concrete structure.

It designates necessary consideration in connection with prospecting for sources of aggregate supply and explains procedure for exploration of deposits when located, methods of sampling, and of tests to be carried on in subsequent laboratory analysis to determine suitability as governed by limitations imposed by the Bureau. Various processes and equipment are either discussed or illustrated in connection with the preparation of natural aggregates for concrete.

Means of batching concrete materials for large dams and for miscellaneous smaller projects are outlined. The fundamentals can apply to all concreting procedure if properly interpreted in order of relative importance. Different types of mixers; the various means by which the concrete is transported from the mixer to the forms; and proper means of placing, are discussed.

The layout of the Bureau Laboratory at Denver, Colorado, as well as both the layout and equipment of field laboratories on certain of the Bureau projects are shown. Examples of typical daily and monthly reports in use by the Bureau are included. The duties and responsibilities of inspectors and their attitude toward the contractor are outlined.

The Concrete Manual may be classed as a valuable contribution to the art, reflecting great credit upon the authors and collaborators.

Compaction of concrete by vibration

K. KAUFMANN, *Beton und Eisen*, Vol. 37, Nos. 16, 17 and 18, p. 263 (Aug. 20, Sept. 5 and Sept. 20, 1938). Reviewed by A. U. THEUER

The author discusses vibrators of the table, surface, internal and form types. General characteristics of German vibrators are described briefly and elementary mathematical principles of vibration are presented. In discussing table vibration, the author relies largely on the results of tests made at the Building Research Station (Great Britain) in order to bring out a number of recommendations for practice. The advantages and limitations of vibrating tables are given in detail. Surface vibration, to which the greater part of these articles is devoted, is discussed at length. On the basis of tests of surface vibrators carried out at Stuttgart, the following principles are enumerated: (1) The workability of concretes must be kept within narrow limits (given in terms of penetration 2 to 5 cm., see D. A. F. E., Vol. 71, p. 57; units are not convertible directly into units intelligible to American practice.) When the concrete is too dry the vibrator will not accommodate itself to the concrete but tends to jump and deliver irregular hammer-like blows. When too wet the vibrator quickly sinks into the mass, collecting water and throwing it up in a spray. (Immersion vibrators are stated to be more suitable for use with wet mixes.) (2) Obviously, poorly selected aggregate gradings were not studied, but experience tends to confirm the view that grading should be within the limits set by standard specifications. Sand contents of pavement concretes should never be less than 45 per cent. When undersanded or when washed aggregates are used at least 2 per cent of fines (0 to .008 in.) should be added to the mix (stated to be necessary for their lubricating effect.) (3) The optimum speed of operation is easily determinable in the job. (Too fast—a large force is required to push the vibrator along, too slow—water collects around the vibrating plank.) A satisfactory mean duration of vibration appears to be 90 secs. per sq. cm. (4) Surface vibration is stated to require a high energy consumption. (5) The most important factor in surface vibration is stated to be its effectiveness with depth. (6) Relative energy expenditures with three different commercial vibrators are given as follows:

Make	*Source of Energy*	*Energy Consumed*
Deprag	Compressed air	0.69 in kg. per sec.
Frisch	Benzol (gas) engine	0.15 in kg. per sec.
Losenhausen	Electric motor	0.04 in kg. per sec.

A large amount of information regarding the effective depths in relation to type of aggregate, kind of foundation, type of criteria of effectiveness (compressive or flexural strength), presence of reinforcing steel, etc., follows. A brief report of a study of form pressures is included.

A description of a second series of tests made in Brandenburg with surface vibrators taking the form of road finishers follows.

General specifications of the commercial types of vibrators investigated are given. The results of these tests briefly listed are: (1) With a speed of vibration regulated to obtain the maximum surface density, the mean speed was found to be 16.8 m. per hr., and the mean energy consumption 6.9 kw. In vibrating 27 cm. slabs the mean work electrically was computed to be 0.137 kwh. per sq. cm., (0.51 kwh. per cu. m.). (2) Optimum wetness—using a 1:6.4 (by weight) mix; sand 49 per cent; aggregate distribution J. (German standards) gave the following results:

Consistency	*Very Stiff*	*Normal*	*Wet*
W/C ratio	0.41	0.51	0.72
Penetrability (Ger. Stds.)	....	1.5	8 cm.
Mean flexural strength after one period of vibration:			
Top surface	37	61	54 kg. per sq. cm.
Base	49	64	54 kg. per sq. cm.

Author here stresses the fact that if a mix becomes too stiff, a point is reached where compressive strength drops off very rapidly and it is better to add a little too much water than too little. (3) In the study of aggregate distribution, the use of a certain percentage of bulky coarse aggregate was found conducive to good compaction. (4) In the study of single and double vibration no particular advantage was noted in the latter case. In practice, however, author recommends a double (two separate periods) vibration. (5) Compaction of two-course concrete slabs, simultaneously, was found to have satisfactory results. (6) Influence of slab thickness on the transverse strength along the lower face was as follows:

(Using same concretes in both cases)

Slab thickness, cm	27	48
W/C ratio	0.51	0.51
No. of vibrations	1	2
Total time, seconds	277	586
Mean flexural strengths:		
Along base, kg. per sq. cm.	61	48
Along surface, kg. per sq. cm.	64	65
Ratio	0.95	0.82

In the case of the thicker slab many large pores formed at heights of 20 to 26 cm. above the base. (7) Compressive strength studies made on cubes cut from slabs again brought out the fact that concrete with large aggregate particles is particularly suited to vibration. Cube strengths do not correlate well with flexural strengths.

Author concludes from these studies that surface vibration is an effective method of obtaining high strength concretes. However, in operation, experience and caution become essential. With road finishers, particuiarly, care must be taken to obtain the right height of setting of the vibrating plank. Set too high the effective depth is decreased, too low the vibration is largely dampened.

Passing on to internal vibrators the author again stresses the importance of workability. (For efficient vibration the penetrability by German Standard Method is stated to be between 3 and 10 cm.) Aggregate gradings, size of mass vibrated and the presence of reinforcement are factors influencing a proper choice. The author states that direct observation in the field will soon enable an operator to determine

the best workability for each specific case. He further states that present grading practices be followed, and stresses the importance of a certain percentage of fines (0 to 0.008 in.) in the sand. To obtain a dense closed surface great precaution must be observed on the job. With insufficient fines, water will be repelled from the mass and form channels along the forms. If the mix is too wet the surface will appear oversanded and will be both weak and porous. The use of admixtures to supply a deficiency of fines may prove beneficial in many cases. The vibrator should never approach nearer to the forms than 5 cms. Along the base of a mass, it being frequently impossible to immerse the vibrator deep enough for full effectiveness (friction between forms and concrete is often greater than the friction in the mass), a richer, wetter mix or hand tamping should be resorted to. The power consumption of the Losenhausen vibrator is stated to be approximately 0.07 kwh. when idling and from 0.10 to 0.20 kwh. when in use.

Form vibrators are treated very briefly. They are not considered to be adaptable except as an auxiliary means in compacting masses such as are generally found in actual construction. They are effective as a means for obtaining a good surface, particularly in inaccessible places difficult to reach with other types.

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Maintenance and Repair of Bridges*

By Conde B. McCullough†

SYNOPSIS

Maintenance costs for highway bridge structures are affected by a number of factors—original design adequacy, traffic burden, local climatic conditions and the character of the stream itself all play an important part. In this paper, those major items of expense encountered in the maintenance of timber structures, steel structures and structures of plain and reinforced concrete are discussed. Particular emphasis is placed upon the maintenance of reinforced concrete in areas of low temperature and heavy ice and snow. Structures built in that portion of Oregon east of the Cascade Range are discussed. This is an area where many cycles of freezing and thawing are experienced each year. The Oregon Department has developed a specialized technique for waterproofing handrails, balustrades and other exposed structural portions, and is also specifying a moderate-heat cement for utilization in this area. Section 5 of the paper reports the average unit costs for those items of expense ordinarily encountered in the maintenance of the various types of highway bridge construction, and the sixth and final section treats of the effect of design details on bridge maintenance expense and the development of special low-maintenance structural types.

1—INTRODUCTION

The maintenance of engineering construction is a function which has hitherto received altogether too little attention on the part of engineers and others in charge of public and private works.

It is necessary, of course, that such structural works be intelligently designed and painstakingly wrought to finished form, but an adequate design and construction procedure is not sufficient within and by itself. It goes only part of the way. Finished construction must, in

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so far as is economically feasible, be protected not only against those natural ravages of time such as corrosion, disintegration and decay, but also from those external agencies which are, to a certain extent, accidental, among which may be mentioned erosion, impact, fire and flood. The technique of maintenance is complicated and involved. In fact, it may be stated that the problems encountered often call for a more intimate knowledge of design practice, materials technique, and construction procedure than any other field of engineering endeavor.

Restricting the discussion to highway bridge types, the extent and character of maintenance required is vitally affected by a number of factors among which may be mentioned: 1) the adequacy of the original design; 2) the traffic burden; 3) local climatic conditions; and 4) the nature and character of the stream involved.

Original design adequacy is especially involved in the case of those bridges which have been taken over or "inherited," as it were, from the local (county or townships) rural road systems. The deck and stringer systems for these structures are generally inadequate for modern highway transport service; and the needed schemes for partial replacements or temporary reinforcing often tax the skill of the designer to the utmost. The traffic service which the structure in question will be called upon to render during the balance of its service life also determines, in large degree, the extent to which reinforcements or partial replacements are necessary. Not only in the case of these old structures, but also for the modern highway bridge, the traffic burden operates to place certain inferior limits upon the expenditures necessary. On high-speed arterials, impact stresses must be reduced to a minimum which requires intensive maintenance of the roadway deck and the junction between the structure and its approach. For highways carrying a large volume of commercial transport, special attention must be paid to the protection of handrailings and main truss members. Local climatic conditions, as above stated, are also determining factors in many instances. The frequency of timber replacement because of decay, the maximum interval between repaint jobs on structural steel, the necessity for protection of concrete surfaces (particularly horizontal surfaces) against weathering or disintegration, and the need for protection against flood and erosion are all wholly or in part functions of climate. The fourth factor hereinabove enumerated—"the nature and character of the stream crossing,"—affects maintenance expense in that it operates in large degree to place inferior limits on the expenditures necessary for bank protection, streambed control, revetment work and other maintenance features of like character. In general, it may be said that those structures which

have been the most adequately designed both for the traffic passing over them and the waterway passing under them will be the least expensive to maintain, but even the most intelligently designed and carefully executed of bridge structures can not be said to be entirely "*maintenance free.*"

The maintenance problems encountered in highway bridge practice tend naturally to group themselves in accordance with the basic type of material used. The discussion which follows will therefore be segregated to consider in turn: a) timber bridges; b) steel structures; and c) bridges of plain nad reinforced concrete.

2—TIMBER STRUCTURES

The principal construction types utilizing this material are trestle structures and truss spans. For the timber trestle the major maintenance item consists of deck renewals. These occur at intervals of from six to ten years, depending upon climatic conditions, location and traffic density, and depending further upon the type of wearing or protective surface employed. The light plank decking of a former decade has proved itself entirely inadequate for the heavy motorized transport of today. The Oregon Highway Department has developed a type of timber deck consisting of a 4 x 12 in. transverse layer covered with a longitudinal running strip 2 x 2 in. in dimension. These running strips are either of cedar or of treated fir. This deck has generally been laid with a ¾-in. spacing between the planks to afford adequate ventilation, and the upper surface coated witn a hot asphalt mastic. Fig. 1 indicates a typical trestle bridge which has just been redecked in this manner. The 2 x 2 in. longitudinal strips have been installed but the asphalt mastic covering has not been placed. Fig. 2 indicates the operation of replacing the plank subdeck. Any decayed or defective stringers are generally replaced during this operation. Fig. 3 indicates the application of the tar mastic over the 4 x 12 in. subdecking and the partial placement of the 2 x 2 in. longitudinal running strips while Fig. 4 indicates the character of the traffic which it is many times necessary to carry during replacement operations.

Among the other major items encountered in the maintenance of timber trestles may be mentioned the redriving of piling and the replacement of caps, sills, posts and stringers. In general, it is necessary to perform the bulk of this work under traffic, and to work in such manner as to cause the minimum of traffic delay and inconvenience. Other maintenance items include the tightening of bolts, the replacement of decayed or damaged handrails, and the cleaning and

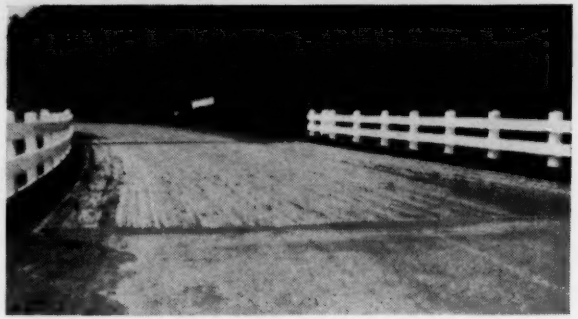
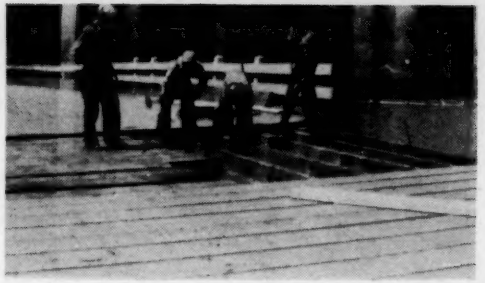


FIG. 1 (TOP LEFT)—PLACEMENT OF A 2 X 2 IN. LONGITUDINAL TIMBER WEARING SURFACE FOR TIMBER TRESTLE BRIDGE DECKS

FIG. 2 (TOP RIGHT)—RENEWING SUBDECK ON A TIMBER TRESTLE BRIDGE—DEFECTIVE OR DECAYED STRINGERS ARE REPLACED



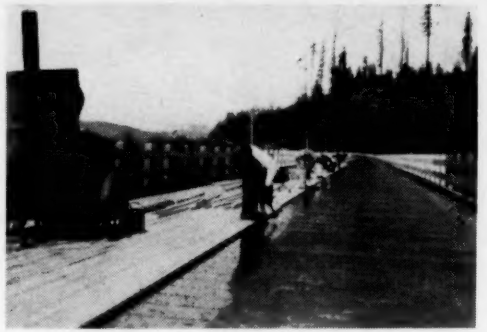


FIG. 3 (BOTTOM RIGHT)—PLACEMENT OF A TWO-PLY DECK AS A MAINTENANCE MEASURE. LOWER DECK OF 4 X 12 PLANK WITH OPEN JOINTS COATED WITH HOT ASPHALT OR TAR PRIOR TO THE PLACEMENT OF THE 2 X 2 LONGITUDINAL WEARING SURFACE

FIG. 4 (BOTTOM LEFT)—HIGHWAY BRIDGE MAINTENANCE MUST GENERALLY BE DONE UNDER TRAFFIC

FIG. 5—A TYPICAL HOUSED TIMBER TRUSS. THE MAINTENANCE ON THIS TYPE OF CONSTRUCTION IS LESS THAN ON TRESTLE STRUCTURES BECAUSE OF THE EFFECT OF THE HOUSING IN PROTECTING STRUCTURAL MEMBERS FROM DECAY

repainting of such portions of the structure at intervals varying between two and four years.

The other principal type of timber structure encountered on Oregon's highways is the timber truss span. Most of these structures have been housed as a protection from the disintegrating action of the weather, as indicated in Fig. 5. Maintenance expenditures for this construction type include deck and handrail maintenance which involve much the same procedure as for timber trestles except that the roadway decks on the covered wooden truss spans appear to require less frequent renewal. In addition to deck and handrail maintenance, timber trusses require a periodic tightening of all bolts and splice connections, and an adjustment of vertical rods to restore any loss in camber due to shrinkage. Less frequent are those maintenance items involving the replacement of floor beams and the renewal or reinforcement of decayed truss members.

Fig. 6 illustrates the reinforcement of an end post on an uncovered timber span in Eastern Oregon. This is an example of reinforcement necessitated by a general increase in the load concentrations carried rather than by decay in the members involved.

In the early periods of highway construction, it was necessary to build some sections on alignment which was recognized to be temporary. On such sections untreated timber was universally used in the structures. As these sections are eliminated, a more permanent type of structure is favored. Timber used in structural work is given a preservative treatment and a composite type of trestle using treated

FIG. 6—THE REINFORCEMENT OF AN END POST OF AN OLD TIMBER TRUSS SPAN. THIS TYPE OF REINFORCEMENT IS NECESSARY IN MANY CASES BECAUSE OF THE INCREASED LOAD CONCENTRATIONS WHICH THESE OLD-TIME STRUCTURES ARE NOW CALLED UPON TO CARRY

timber for substructure, caps and stringers, and reinforced concrete for the deck has been developed. By proper attachment between the concrete deck and the timber stringers, a T-beam action has been obtained which stiffens the structure and decreases the deflection. In the last six years approximately half of the total length of structures built in Oregon has been of this type of construction. These structures have given admirable service and the maintenance costs have been quite low. This type of structure is limited to spans of 30 feet or less, and has been found most adaptable where foundation conditions are such that piles are necessary for the support of any type of structure.

3—STEEL CONSTRUCTION

The inadequacy of the original design to carry modern highway transport loadings is one of the principal features militating toward increased maintenance expense for this type of structure. Defects of this character have been remedied in many instances by a system of carefully designed reinforcement, utilizing additional looprod members and welded plates and angles for floor beams and other members susceptible to this type of attack. For the modern steel bridge, the chief maintenance expense is that for painting, and the annual cost is obviously determined largely by the periodicity of repainting operations. Structures exposed to the salt-laden atmosphere of the coastal regions require painting at periods varying between two and four

FIG. 7—THE PLACEMENT OF SHEAR STRIPS OR COLLISION RAILS ABOVE THE HANDRAILING OF AN OLD STEEL TRUSS AS A PROTECTION AGAINST IMPACT FROM OVERHANGING TRUCK BODIES. ITEMS OF THIS CHARACTER ARE NECESSARY IN MANY INSTANCES BECAUSE OF THE WIDTH, WEIGHT AND HIGH SPEED OF MODERN TRUCK TRAFFIC

years, while those in the arid areas of Eastern Oregon have rendered satisfactory service without repaint jobs for intervals as long as eight to ten years.

In addition to painting operations and such normal periodic maintenance as is necessary for this type of structure, modern traffic speeds and load concentrations in certain instances demand special protection, particularly in the case of old structures. Fig. 7 indicates the placement of shear rails above the handrail of a steel truss span in order to protect the truss web system against possible damage from collision impact due to overhanging truck bodies. Before these rails were placed, several of the truss members had been badly damaged.

4—CONCRETE CONSTRUCTION

Maintenance and repair expense for concrete cosntruction is generally a very small percentage of its original cost. Principal among the items which have been necessary in this category are the repair and replacement of damaged handrails, the wire brushing and refinishing of rails to provide greater night visibility, and revetment work and pier protection for stream conditions necessitating such items. In most instances, the damage to handrailings and curbs is, in part, the result of inadequate roadway widths, but in many cases high speed and reckless traffic have operated to cause considerable damage of this character even on the most amply designed of modern construction.

In addition to the above routine maintenance necessities, some concern has been occasioned by the deterioration or excess weathering of certain concrete surfaces. The worst examples of such weathering

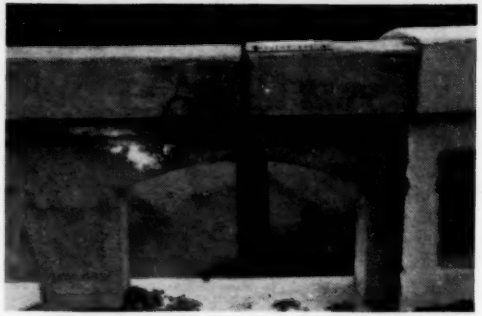


FIG. 8 (TOP LEFT)—HANDRAIL POST ON EARLY CONCRETE BRIDGE, INDICATING SURFACE DISINTEGRATION OR WEATHERING. THIS TYPE OF WEATHERING ACTION HAS BEEN OBSERVED IN A NUMBER OF THE OLDER STRUCTURES SO LOCATED AS TO BE SUBJECTED TO REPEATED CYCLES OF FREEZING AND THAWING DURING THE WINTER MONTHS

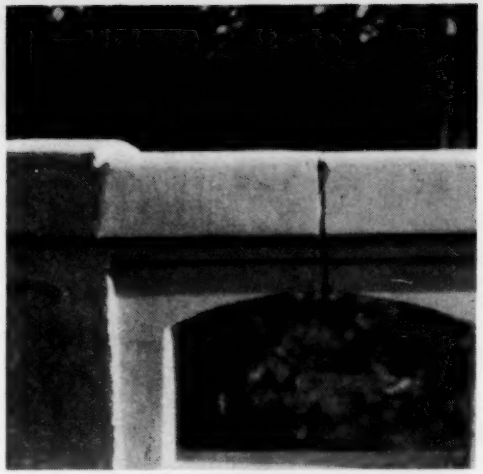


FIG. 9 (BOTTOM LEFT)—SURFACE DISINTEGRATION AT EXPANSION JOINT IN HANDRAIL

FIG. 10 (RIGHT)—SAME STRUCTURE SHOWN IN FIG. 9, AFTER THE DEFECTIVE MATERIAL HAS BEEN CHIPPED AWAY AND REPLACED, AND THE SURFACE PROTECTED BY REPEATED APPLICATION OF HOT RAW LINSEED OIL AND A FINAL LEAD AND OIL PAINT COAT

action have been observed in that portion of Oregon east of the Cascade Range where many cycles of freezing and thawing are experienced each year. In the main, this weathering is confined to exposed surfaces such as handrail copings and roadway or sidewalk curbs. There are, in general, two distinct types of this weather deterioration which, while closely related, are apparently the result of different primary causes. In the first type, wide cracking occurs with some of the individual cracks opening up as much as one-quarter inch or wider. In general, the concrete in the vicinity of the individual cracks is found to be fairly hard and sound. It is thought that this type of deterioration is primarily the result of a latent unsoundness in the cement, probably an excess of free lime. The second and perhaps more serious type of weathering appears to start with a maze of hair-line cracks which have a tendency to parallel the longer dimensions of the members. These hair checks permit moisture to enter, and subsequent freezing and thawing appears to complete the disintegration of the material affected. The entire body of the concrete becomes soft and lifeless so that it can be readily crumbled between the fingers. Deterioration of this second class has become quite noticeable on a considerable number of structures in the Eastern Oregon district, and an investigation has been in progress for the past two years looking toward the determination of the underlying cause or causes. The highway department files at present contain a complete record of the source of each of the aggregates used in the construction of these structures, and in each case the original source has been resampled and investigated. Most of these structures were built prior to the adoption of the sodium sulphate test for aggregate soundness, and while no definite conclusions can be drawn at this time, there appears to be a marked correlation between the condition of the concrete in the field and the results of the soundness test on the sand and coarse aggregate employed.

This department has also made laboratory specimens using the actual aggregates employed in these structures, and has subjected these specimens to approximately 160 cycles of freezing and thawing. The data for these tests have not as yet been analyzed but some instances have been noted wherein specimens made from aggregates which show unfavorably under the soundness test have also shown marked disintegration under repeated freezing and thawing. It does not appear at this time, however, that the correlation between these laboratory tests and the actual field behavior of the structures corresponding thereto will be very high. This type of disintegration on a handrail post of one of Oregon's earlier structures is indicated in Fig. 8.

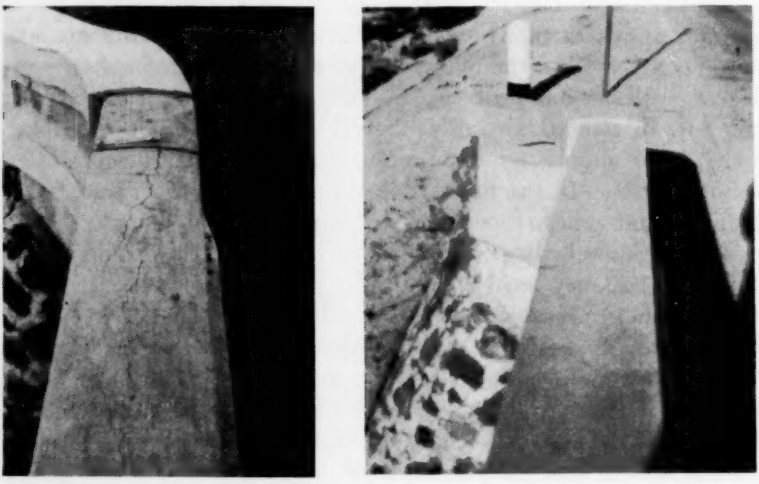


FIG. 11 AND 12—LONGITUDINAL CRACKING IN THE TOP OF A CONCRETE BALUSTRADE (LEFT) AND SAME BALUSTRADE (RIGHT) AFTER THE APPLICATION OF WATERPROOFING

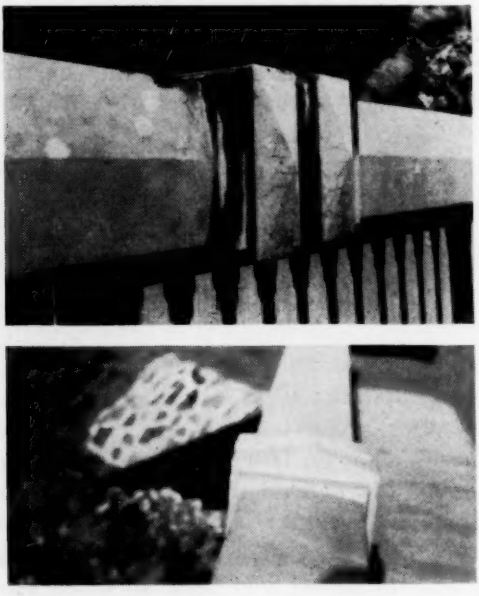


FIG. 13—CONCERTE BALUSTRADE ON TYPICAL COMPOSITE TRESTLE CONSTRUCTION BEFORE AND AFTER WATERPROOFING TREATMENT

During the past year the highway department has repainted and waterproofed a number of those structures which indicated the most pronounced deterioration of this class. Essentially, this work consisted of chipping out the unsound concrete and patching back the members to their original sectional dimension. After thoroughly cleaning both old and new concrete, the surface was treated with a brush coat of hot raw linseed oil mixed half and half with turpentine. This prime coat was followed with one or more additional coats of hot raw linseed oil until the concrete refused to absorb any further waterproofing material. The work done thus far indicates a penetration of slightly more than one-quarter inch as evidenced by the discoloration of the concrete, determined from sampling. The linseed oil applications were followed by a coat of lead and oil paint, the color being selected to match as closely as possible the original concrete surface. The linseed oil waterproofing treatment resulted in a somewhat objectionable dirty brown color which furnished the reason for the final paint application. The waterproofings thus placed have only been in service during the current season and it is obviously far too early to make any definite prediction as to their service value. At the last inspection, however, they appeared to be standing up in first-class shape, and to present a satisfactory appearance. Fig. 9 is a view of a concrete handrail showing the original disintegration while Fig. 10 indicates this same portion of the structure after the application of the waterproofing treatment. Figs. 11 and 12 indicate a typical handrail cap before and after waterproofing. Fig. 13 is another view indicating the handrail cap on one of the state's composite (timber and concrete) structures in Eastern Oregon before and after the waterproofing treatment.

It is thought that the initial curing of the concrete has had considerable effect upon the formation of the hair-line checks from which this type of weathering or deterioration apparently begins. Most of the concrete construction in Eastern Oregon is, of necessity, performed during the summer months. The temperature is high and the humidity low, rendering curing a rather difficult process, especially with a concrete which hardens rapidly. Although no definite conclusions can be stated, the state's investigation thus far indicates that the greatest trouble has been experienced with cements which were high in tricalcium aluminate and correspondingly low in iron compounds. Whether this is due to any quality or property inherent in the cement itself or to the increased difficulty in proper curing is a point concerning which no definite statement can be made at this time.

TABLE 1—ITEMS COMMON TO UNTREATED TIMBER TRESTLE SPANS AND ALL BRIDGES WITH UNTREATED TIMBER DECKS

Maintenance Items	Budget Estimates Cost	Unit
1. Clean timber handrail	$ .04	Lin. ft. of handrail
2. Clean and repaint handrail one coat (includes creosoting felloe guard)	.15	Lin. ft. of handrail
3. Clean and repaint handrail two coats (includes creosoting felloe guard)	.25	Lin. ft. of handrail
4. Renew timber handrail (includes felloe guard, drip board, painting, and removing old handrail)	2.00	Lin. ft. of handrail
5. Replace drip board	.15	Lin. ft. of drip board
6. Renew felloe guard	.25	Lin. ft. of fel. guard
7. Miscellaneous handrail renewals	100.00	Per M-FBM
8. Tighten bolts (18′ to 24′ roadways) average	.15	Lin. ft. of span
9. Place additional stringers	15.00	Per stringer
10. Redecking (general)	55.00	Per M-FBM
11. Redeck with 4x12 & 2x2—20′ roadway	8.00	Lin ft. of span
12. Redeck with 4x12 & 2x2, and widening 20′ to 24′ roadway (replacing old rail)	10.00	Lin. ft. of span
13. Redeck with 4x12 & 2x2, and widening 20′ to 24′ roadway (renewing timber handrail)	12.00	Lin. ft. of span
‡14. Frame bent old piling substructure	60.00	Per M-FBM
Frame bent old piling substructure	6.00	Per Ft. Hgt. of bent
‡15. Replace caps (12x12—28)	25.00	Per cap
‡16. Replace bulkheads	65.00	Per M-FBM
Replace bulkheads	15.00	Per Ft. Hgt.

‡Average costs for roadways from 18 to 24 feet in width.

TABLE 2—ITEMS COMMON TO UNTREATED TIMBER TRUSS SPANS
(Roadways 18′ to 20′)

Maintenance Items	Budget Estimates Cost	Unit
1. Take up camber	$1.00	Lin. ft. of span
Take up camber	3.50	Per rod
2. Tighten bolts	.25	Lin. ft. of span
3. Repaint housing exterior two coats white	3.00	Lin. ft. of span
Repaint housing exterior two coats white	.05	Per sq. ft.
4. Whitewash interior	1.00	Lin. ft. of span
Whitewash interior	.02	Per sq. ft.
5. Restain shingle roof	1.00	Lin. ft. of span
Restain shingle roof	.02	Per sq. ft.
6. Creosote truss members (brush coat treatment)	1.00	Lin. ft. of span
Creosote truss members (brush coat treatment)	.02	Per sq. ft.
7. Fill season checks with asphalt	.20	Lin. ft. of span
8. Replace steel splice connections	65.00	Per connection
9. Replace timber splice connections	50.00	Per connection
10. Replace corbel blocks	15.00	Per corbel block
11. Replace floor beams	100.00	Per floor beam
12. Retimber trusses	90.00	Per M-FBM
13. Miscellaneous timber truss replacements	200.00	Per M-FBM

TABLE 3—ITEMS COMMON TO STEEL SPANS

Maintenance Items	Budget Estimates Cost	Unit
1. Cleaning and spot painting steel (light cleaning)	$1.25	Per Ton
2. Cleaning and spot painting steel (average cleaning)	3.50	Per Ton
3. Cleaning and spot painting steel (heavy cleaning)	7.00	Per Ton
Note: Items 1, 2, and 3, are for cleaning with wire brushes and hand scrapers.		
4. Sandblasting steel (heavy cleaning)	15.00	Per Ton
5. Repainting steel trusses (red lead)	2.50	Per Ton per coat
6. Repainting steel trusses (black graphite)	2.00	Per Ton per coat
7. Repaint steel handrail white (lattice type)	.30	Lin. ft. railing
Note: The following formulas give approximate weight of steel in pounds per lineal foot of truss for 20-foot roadway, H-15 loading.		
Pony trusses — $W = 5L + 525$		
Thru trusses — $W = 4.5L + 460$		
W = Weight per lineal foot		
L = Length of span in feet		

In order to secure a concrete which will afford greater time for curing, this department has adopted a moderate heat cement (Federal Bureau of Procurement Specification SS-206) for this district. Close inspection of the surfaces on those structures which have been built with this latter type of cement indicates that, to date, almost all of the hair checking has been eliminated. It is hoped that this modification will result in a concrete which is more nicely adapted to this rather frigid region. It is further hoped that the results of the investigations under way in the Oregon laboratories under the direction of Glenn S. Paxson, Bridge Engineer for the department, will result in the development of a tighter specification for aggregates, and a more rigid control of field manipulation, placement and curing.

5—COST DATA

As stated, unit costs for bridge maintenance vary with a number of factors. The adequacy of the original design, the character and density of traffic, climatic conditions, and the nature of the stream crossing itself, all have their effect. As a rough guide for estimating purposes, Tables 1 to 6 indicate the items common to each type of highway bridge construction, and the average unit costs which are employed for budget estimates in the absence of better data for the specific instances involved.

These tables are based upon the following average unit prices, except where otherwise noted:

Untreated timber (fir) F. O. B. job	$25.00 per MBM
Untreated timber (cedar) F. O. B. job	30.00 per MBM
Treated timber F. O. B. job	70.00 per MBM
Concrete in place	25.00 per cu. yd.
Reinforcing metal in place	0.06 per lb.
Paint for timber structures F. O. B. job	2.00 per gal.
Creosote for brush coat application, F. O. B. job	0.35 per gal.
Cold water paint (whitewash) F. O. B. job	0.40 per gal.
Shingle stain, F. O. B. job	1.00 per gal.
Paint for structural steel F. O. B. job	2.00 per gal.

Labor:

Bridge foreman	165.00 per month
Bridge carpenters	0.75 per hour
Carpenters' helpers	0.65 per hour
Bridgemen	0.60 per hour
Common labor	0.50 per hour

TABLE 4—ITEMS COMMON TO CONCRETE STRUCTURES

Maintenance Items	Budget Estimates Cost	Unit
1. Wire brush concrete handrail	$ .10	Lin. ft. handrail
2. Patch and refinish concrete handrail	.50	Lin. ft. handrail
3. Replace broken precasts	1.50	Per precast
4. Replace broken concrete handrail	5.00	Per lin ft.
5. Replace bridge reflectors	3.00	Per reflector
6. Mop coat deck with emulsified asphalt and sand	.10	Per sq. yd.
7. Miscellaneous minor items—clipping form wires and painting, etc.	Lump sum	Per bridge

TABLE 5—ITEMS COMMON TO COMPOSITE CONSTRUCTION AND OTHER CREOSOTED TIMBER SPANS

Maintenance Items	Budget Estimates Cost	Unit
1. Re-oil timber spindles	$ .10	Lin. ft. of handrail
2. Replace timber spindles	.50	Per spindle
3. Tighten bolts (20′ to 27′ roadways)	.10	Lin. ft. of span

TABLE 6—ITEMS COMMON TO ALL BRIDGES

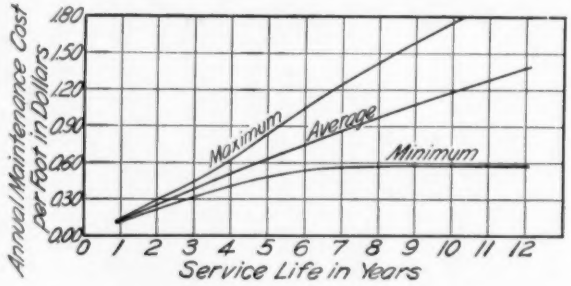
Maintenance Items	Budget Estimates Cost	Unit
1. Underpin concrete footings	$50.00	Per cu. yd.
2. Rip rap (dry, dumped)	3 .00	Per cu. yd.
3. Rip rap (dry, hand-placed)	4.00	Per cu. yd.
4. Piling revetment (sheeted and braced, but not backfilled)	10.00	Lin. ft. of revetment
5. Piling revetment (sheeted, braced and backfilled with brush and rock)	20.00	Lin ft. of revetment
6. Basket rip rap	10.00	Per cu. yd.
7. Rock-filled log cribs	8.00	Per cu. yd.
8. Miscellaneous minor items, cutting brush, removing drift, cleaning out channel, and repairs to slope protection	Lump Sum	Per bridge

Fig. 14 to 17 have been plotted from compiled data from the Oregon maintenance records, and indicate the average annual maintenance expense per foot to date as a function of service life in years for the four most commonly used bridge types.

Fig. 14 is for timber trestle structures. The abscissae represent service life value, as ordinates are plotted the average annual maintenance expense (in dollars per foot). Three curves are indicated, the lower one being for the arid regions of eastern Oregon while the upper two are in the more humid sections of the Cascade Range. The coastal region is the most humid, and as would be expected, shows the largest maintenance cost. All curves slope markedly upward with increasing service life.

Fig. 15 is for timber truss bridges and indicates the same general upward trend but in lesser degree. The affect of humid climatic conditions upon maintenance expense is also plainly indicated.

Fig. 16 is for steel truss and girder spans. The affect of the salt-laden atmosphere of the coastal regions upon metal corrosion is plainly reflected in the high maintenance costs for steel structures on the coast.



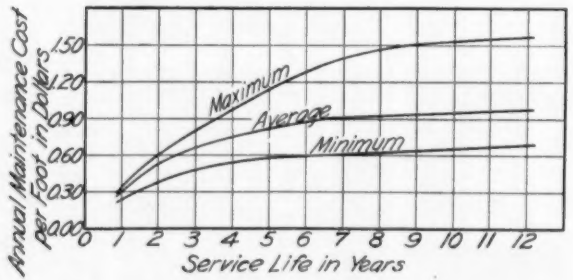
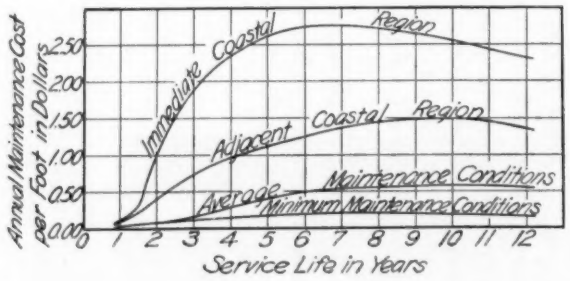


FIG. 14 (TOP)—ANNUAL MAINTENANCE COSTS OF TIMBER TRESTLE BRIDGES, FOR MAXIMUM, AVERAGE AND MINIMUM MAINTENANCE CONDITIONS

FIG. 15 (BOTTOM)—ANNUAL MAINTENANCE COSTS OF TIMBER TRUSS BRIDGES FOR MAXIMUM, AVERAGE AND MINIMUM MAINTENANCE CONDITIONS



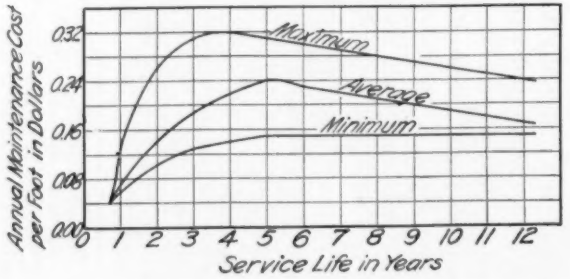


FIG. 16 (TOP)—ANNUAL MAINTENANCE COSTS OF STEEL TRUSS AND GIRDER BRIDGES

FIG. 17 (BOTTOM)—ANNUAL MAINTENANCE COSTS OF CONCRETE BRIDGES FOR MAXIMUM, AVERAGE AND MINIMUM MAINTENANCE CONDITIONS

Fig. 17 is for concrete structures. In this type the maintenance costs are very small and are not materially affected either by age or climatic conditions. This is, of course, not strictly true since the colder temperature of the arid region subjects concrete to a more severe punishment than in the more equable coastal and valley climates. The entire cost for maintenance expense for this structural type, however, is so small as to render a fixed figure per foot sufficiently exact for estimating purposes.

From the foregoing, and from a study of Fig. 14 to 17, inclusive, it would appear that the following assumptions would be sufficiently exact for estimating purposes in the absence of more applicable and accurate data.

Average Annual Cost Per Foot for Structural Maintenance:

a. Timber trestle bridges (See Fig. 14):	
1. Minimum maintenance (arid climate)	$0.60
2. Maximum maintenance (humid climate)	2.00
b. Timber truss bridges (see Fig. 15):	
1. Minimum maintenance (arid climate)	0.70
2. Maximum maintenance (humid climate)	1.60
c. Steel girder and truss bridges (see Fig. 16):	
1. Minimum maintenance (arid climate)	0.20
2. Average maintenance (humid climate)	0.60
3. Maximum maintenance (locations exposed to salt-laden air)‡	1.50
d. Concrete bridges (see Fig. 17):	
1. Average maintenance (any climate)	0.20

‡For extreme conditions immediately adjacent to ocean the annual maintenance for steel structures may average as high as $2.75 to $3.00 per foot (see Fig. 16).

The curves shown in Fig. 14 to 17 were plotted through mean values in order that they might be more readily employed for estimating purposes. Curves plotted from actual maintenance cost records for the various structural types will obviously indicate a slight variation; however, it is felt that these values are sufficiently exact for rough estimates, and when time is not sufficient to permit the utilization of the detailed cost figures given in Tables 1 to 6.

6—THE EFFECT OF DESIGN DETAILS ON BRIDGE MAINTENANCE EXPENSE

As has been stated, maintenance expense of bridges varies with the original design adequacy, traffic burden, local climatic and stress conditions, and other related factors. Another factor influencing maintenance costs is the selection of design details. By careful selection of these features of design, greatly reduced maintenance may be effected. This is particularly important in the case of reinforced concrete construction where the maintenance expense is in itself a rather small item, and even a small annual monetary reduction will operate to effect a comparatively large percentage saving. Typical details of this character are: (1) the joint or transition between roadway slab and bridge deck; (2) details for protection against damage due to traffic collision impact; and (3) details which operate to minimize disintegration or excessive weathering due to snow and ice.

FIG. 18—TYPICAL DISINTEGRATION AROUND THE BASE OF A CONCRETE LAMP POST. EXPOSED VERTICAL SURFACES OF THIS CHARACTER APPEAR TO SHOW THE GREATEST WEATHERING ACTION

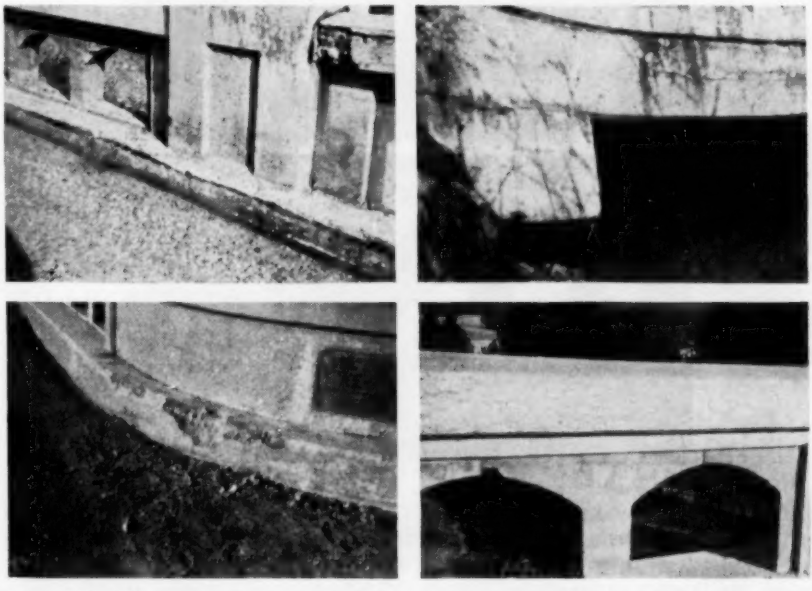


FIG. 19—FOUR VIEWS INDICATING WEATHERING OBSERVED ON OLD CONCRETE STRUCTURES EXPOSED TO REPEATED CYCLES OF FREEZING AND THAWING. THESE DEFECTS ARE BEING CORRECTED BY REMOVAL AND REPLACEMENT OF MATERIAL AFFECTED, AND THE APPLICATION OF A SUITABLE WATERPROOFING.

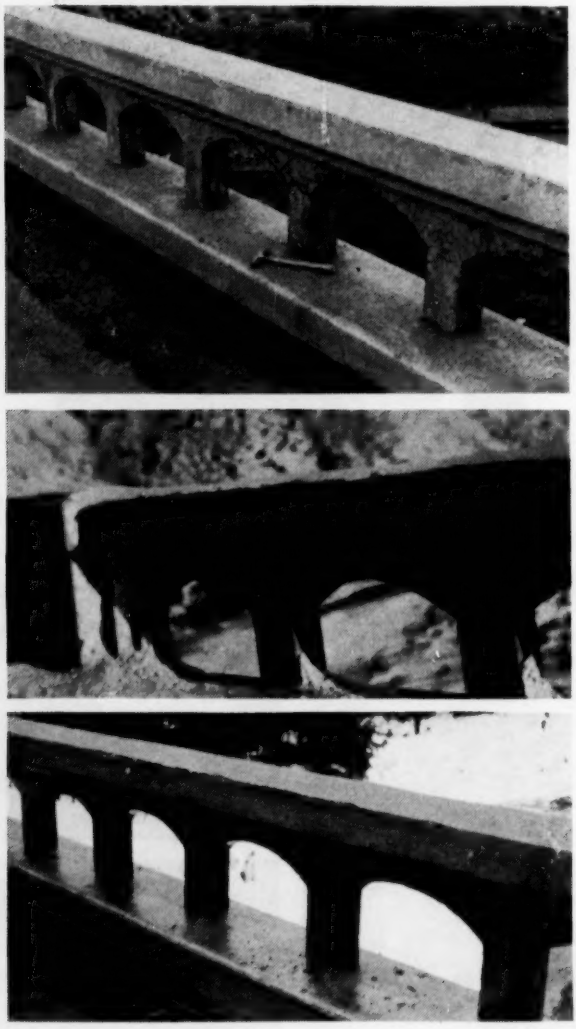


FIG. 20—THREE VIEWS INDICATING TYPICAL DISINTEGRATION OBSERVED IN CONCRETE HANDRAILS EXPOSED TO REPEATED FREEZING AND THAWING CYCLES DURING THE WINTER MONTHS. HANDRAILS AND OTHER HORIZONTAL EXPOSED SURFACES APPEAR TO SUFFER MOST

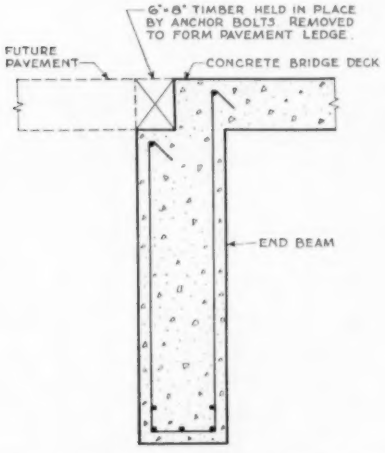


FIG. 21—JOINT OR TRANSITION BETWEEN ROADWAY SLAB AND BRIDGE DECK

Fig. 21 is a detail of the character first above described, and one which has been adopted as standard for the end floor beams of all concrete bridges constructed in Oregon. A ledge or bracket is formed to receive and seat the future pavement slab, and as a temporary measure (before the approach fills have settled sufficiently to permit the placement of pavement) a 6 x 8 in. timber held in place by anchor bolts acts as a temporary buffer, protecting the structure against what otherwise might develop into a shattering impact. It is generally specified that the pavement panel immediately adjacent to the bridge structure be rather heavily reinforced in order to transfer the load over a comparatively wide area in the event of any possible settlement of the approach fills. A detail of this kind appears absolutely necessary if a smooth transition between pavement and structure is to be provided, for regardless of the degree of care exercised, it is impossible to avoid some slight volume change in the embankment material.

Fig. 22 and 23 illustrate details designed for the prevention or lessening of damage due to collision impact. This is an extremely important feature with the heavy high speed motorized transport which utilizes the highways of today. In fact, it may be said that, outside of the surface weathering caused by low temperatures, ice and snow, damage from traffic impact is responsible for the major portion of the maintenance expense for concrete structures. A variety of details suggest themselves in this connection, but Fig. 22 and 23 are typical. FIG. 22 represents a plan and elevational view of a skewed grade

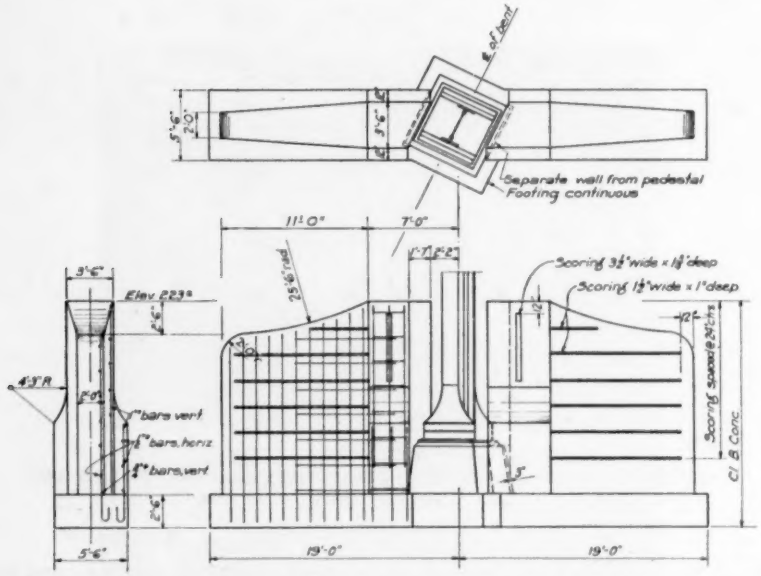


FIG. 22—DETAILS FOR PREVENTION OR LESSENING OF DAMAGE DUE TO COLLISION AT COLUMNS ON GRADE SEPARATION STRUCTURE

separation structure. The columns are protected from traffic collision by means of low walls heavily reinforced and separated from the pedestal, as indicated. Fig. 23 indicates a device utilized to protect the ends of the sidewalk area and the adjacent bridge rails by means of a suitable reinforced concrete buffer fitted with amber flashing traffic lights. This device or similar ones find particular application in construction over urban arterials wherein it is necessary to separate grades and to provide for an inclined ramp leading down and to the right of the structure proper.

The construction of higher and wider curbs and the stiffening up of design requirements for handrailings, both as regards vertical and lateral load capacity, are among the other details of this type which have been found necessary to lessen those items of maintenance expense directly chargeable to traffic collision impact.

In the low temperature areas of the state, particularly in those regions of heavy snowfall, the selection of suitable design details may operate in considerable degree to lessen subsequent maintenance costs. In regions of this character, horizontal surfaces are to be avoided, particularly on handrail copings and other members of comparatively light cross section. Pier caps should also be sloped sharply in order

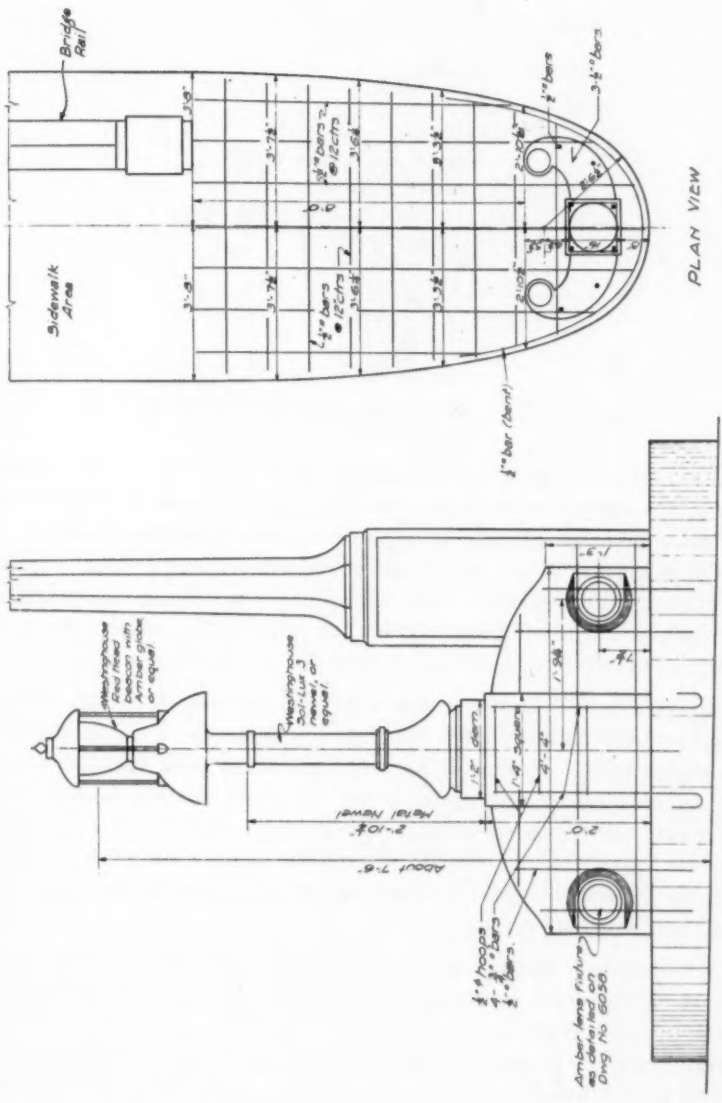

Bridge Rail
Sidewalk Area
PLAN VIEW
END ELEVATION.
Westinghouse Red Head beacon with Amber globe or equal
Westinghouse 301-Lux 3 newel, or equal
Metal Newel
ROOM 7'-6"
Amber lens fixture as detailed on Dwg No 6058

FIG. 23—DETAILS OF REINFORCED CONCRETE BUFFER TO PROTECT ENDS OF SIDEWALK AREA AND BRIDGE RAILS

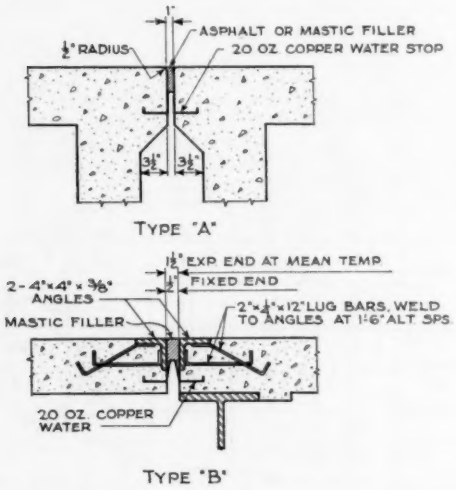


FIG. 24—TWO TYPES OF EXPANSION JOINTS

to drain rapidly, and protected wherever possible by a superstructure overhang, the optimum condition being that in which the accumulation of snow or ice is precluded to the maximum possible degree. In areas of this kind it is also essential to waterproof and protect expansion joints wherever possible. Fig. 24 indicates two types of expansion joints which are protected by a 20-ounce copper flashing in order to preclude the possibility of disintegration due to the entrance of water.

The above are only a few of the many design details which vitally affect subsequent maintenance expense but space will not permit of further elaboration. Suffice it to say that the major factors which operate to increase maintenance costs for reinforced concrete construction are water, frost, and traffic impact, and those structures wherein the greatest degree of protection against these factors is attained, will show the most satisfactory maintenance ledger over the years of their service periods.

Another operation which has proved potent to decrease maintenance expense is the gradual improvement and development of particular structural types for particular localities. An example of this sort is the development in Oregon of the *composite trestle type*. This design, mentioned rather briefly hereinbefore, consists essentially of a treated timber substructure and stringer system supporting a reinforced concrete roadway deck integral therewith. This type, a cross section of which is indicated in Fig. 25, was developed by the Oregon Highway Department in 1930 and 1931, as a result of a series of loading tests

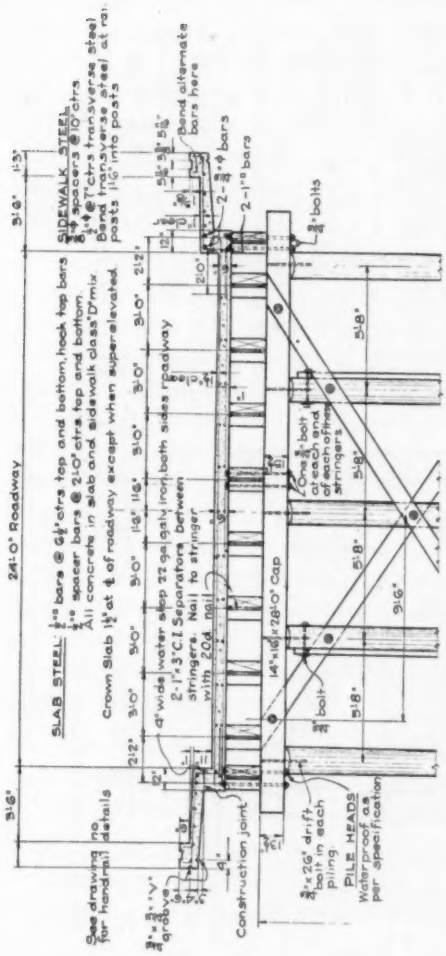

24'-0" Roadway
SLAB STEEL: ½" bars @ 6½" ctrs top and bottom, hook top bars
½" spacer bars @ 2'-0" ctrs top and bottom.
All concrete in slab and sidewalk class "D" mix
Crown Slab 1½" at ℄ of roadway except when super-elevated.
SIDEWALK STEEL:
½" spacers @ 10' ctrs
½" @ 7" ctrs transverse steel
Bend transverse steel at rail posts 16" into posts
4" wide water slop 22 ga galv iron both sides roadway
2-1"x3"C.I. Separators between stringers. Nail to stringer with 20d nail
Construction joint
14"x16"x28'-0" Cap
¾" bolt
One ¾" bolt at each end of each of these stringers
¾" bolts
¾"x26" drift bolt in each piling
PILE HEADS
Waterproof as per specification
2-¾" ϑ bars
2-1" ϑ bars
Bend alternate bars here
See drawing no. for handrail details
¾"x¾" V groove
5'-1"
5'-3"
3'-6"
2'-0"
2'-6"
3'-0"
1'-4½"
5'-8"
9'-6"

FIG. 25—TYPICAL CROSS SECTION OF COMPOSITE TRESTLE

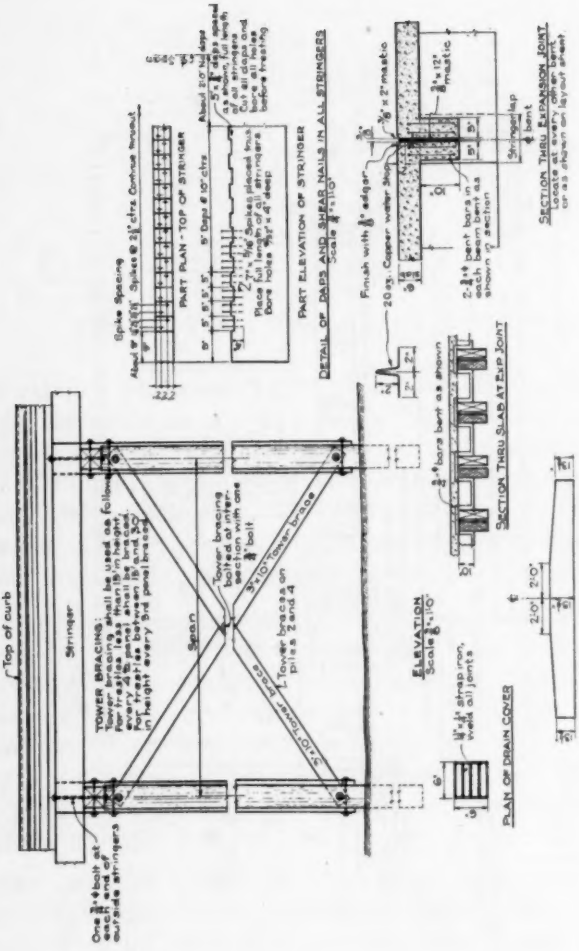

Top of curb
Stringer
Span
TOWER BRACING:
Tower bracing shall be used as follows:
Where trestle height is 15 ft. or less, shall be braced every 40 ft. panel shall be braced. For trestle between 15 and 30 ft. in height every 3rd panel is braced.
Tower bracing bolted at inter. section with one 3/4" bolt
3"x10" tower brace
Tower braces on piles 2 and 4
3"x10" tower brace
One 3/4" bolt at each end of outside stringers
PART PLAN - TOP OF STRINGER
Spike Spacing
Spikes @ 24" ctrs. Centre front
Spikes @ 12" ctrs
5"x7" cap spaced as shown, full length of all stringers
Place 2"x6" Soaked blocked thick between stringers
Bore holes 3/4" x 4" deep
PART ELEVATION OF STRINGER
DETAIL OF CAPS AND SHEAR NAILS IN ALL STRINGERS
Scale 1"=1'0"
Finish with 1/2" edger
20 ga. Copper water stop
3/8" x 2" mastic
3" x 12" mastic
2-3"x8" bent bars on each beam bent as shown in section
Stringer cap
Bent
SECTION THRU EXPANSION JOINT
Loose plate as shown or as shown on layout sheet
2-3"x8" bent bars as shown
SECTION THRU SLAB AT EXP JOINT
ELEVATION
Scale 1/8"=1'0"
1/4" strap iron, weld all joints
PLAN OF DRAIN COVER
CROWN IN 16"x16" CAP
Cut as shown

FIG. 26—TYPICAL ELEVATION AND DETAILS OF COMPOSITE TRESTLE

described in Highway Department Technical Bulletin No. 1 entitled "Loading Tests on a New Composite Type Short-Span Highway Bridge, Combining Concrete and Timber in Flexure." This investigation comprehended the test to destruction of T-beams composed of timber webs and reinforced concrete flanges connected at the junction in several ways. These tests rather conclusively demonstrated the fact that the ultimate strength of a composite T-beam of this character suitably connected at the junction plane and suitably designed, was at least twice that for the same materials and the same dimensions independently employed. It was further demonstrated that the deflection of such a composite beam would not be more than 25 per cent of the corresponding deflection for the same sizes and materials independently employed.

As hereinbefore stated, several types of shear connections were tested but the results of the investigation were such as to indicate that the use of combined spikes and daps produced the best results, all things considered. The utilization of metal pipe dowels was also found to be entirely satisfactory, but this detail has been discarded in favor of the former method because of its greater economy in first cost. These composite or synthetic T-beams were subjected to repeated or alternated loads and to variations in temperature. It was found that temperature stresses induced by a variation from 15° to 70° F. were sufficient in magnitude to warrant a definite provision for them in the design. It was further developed, however, that this feature could adequately be taken care of through a proper design of the shear connection. It was also found that the ordinary "transformed section" formulas yield results which, within working limits, are sufficiently accurate for all practical design purposes.

Fig. 26 is an elevational view of a composite trestle structure of this type and also indicates in detail the type of shear connection known as the "dap and spike type."

Fig. 27 and 28 are construction views, the upper view in Fig. 27 shows the pipe dowel type while the lower view indicates the dapped type of shear connection. The upper view in Fig. 28 plainly indicates the placement of spikes in the dapped stringer tops and the slab reinforcement prior to the placement of concrete. The pipe dowel type of shear connection is indicated in the lower portion of Fig. 28.

Fig. 29 is a deck view of a finished structure of this type, and Fig. 30 a close-up view showing the type of handrailing utilized, the posts and balustrade being of reinforced concrete with oiled or varnished timber spindles. Fig. 31 is a composite superstructure supported by rein-

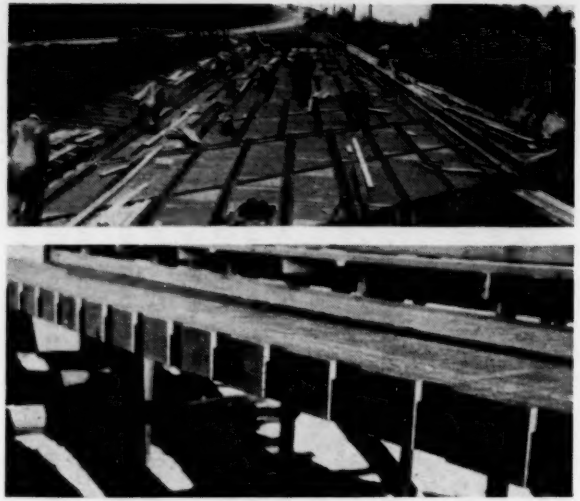


FIG. 27 (ABOVE)—CONSTRUCTION VIEWS OF COMPOSITE TYPE STRUCTURE. (TOP) STAGE OF CONSTRUCTION PRECEDING THE PLACEMENT OF THE CONCRETE DECK. THE PIPE DOWEL SHEAR CONNECTIONS ARE PLAINLY INDICATED. (BOTTOM) THE DAPPED TYPE OF SHEAR CONNECTION. ALL FRAMING FOR DAPS IS DONE PRIOR TO TREATMENT

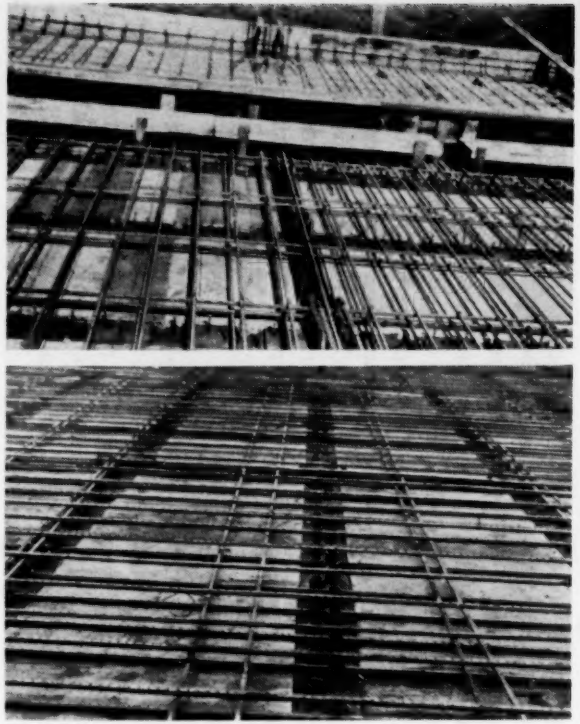


FIG. 28 (RIGHT)—TWO CONSTRUCTION VIEWS OF COMPOSITE TRESTLE DECKS. (TOP) THAT STAGE OF CONSTRUCTION IMMEDIATELY PRECEDING THE PLACEMENT OF THE CONCRETE DECK. THE SPIKES AND DAPS WHICH FORM THE SHEAR CONNECTION BETWEEN THE DECK AND STRINGER SYSTEM ARE PLAINLY INDICATED. THE LOWER VIEW INDICATES THE UTILIZATION OF PIPE DOWELS FOR THE PURPOSE OF SHEAR TRANSFER

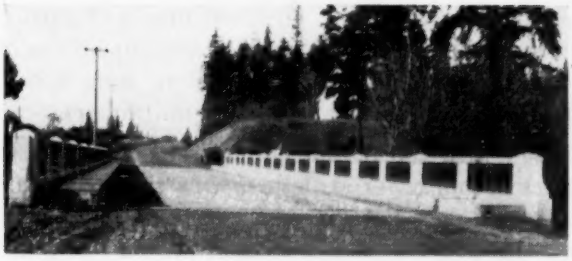


FIG. 29—DECK VIEW OF A COMPOSITE TRESTLE

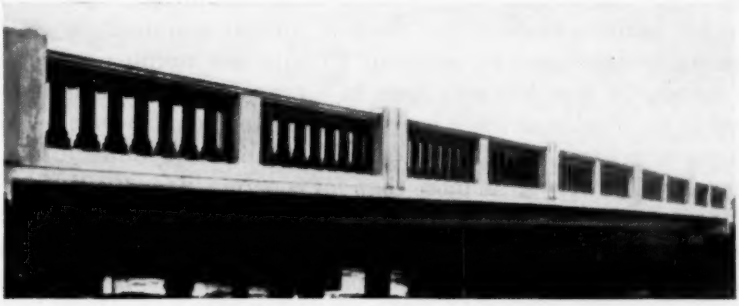


FIG. 30—VIEW OF COMPOSITE TIMBER HANDRAIL. THE POSTS AND BALUSTRADES ARE OF REINFORCED CONCRETE WITH OILED OR VARNISHED TIMBER SPINDLES. THIS TYPE OF HANDRAILING IS UTILIZED IN CONNECTION WITH THE COMPOSITE CONSTRUCTION ILLUSTRATED IN FIG. 29

FIG. 31—A COMPOSITE SUPERSTRUCTURE SUPPORTED BY REINFORCED CONCRETE PEDESTAL PIERS. THIS STRUCTURE IS OF PARTICULAR INTEREST BECAUSE OF THE EXTRAORDINARY SPAN LENGTHS AND DIMENSIONS OF THE TIMBER STRINGERS. THE INDIVIDUAL SPANS ARE 40 FEET AND THE STRINGER SYSTEM CONSISTS OF EIGHT LINES OF 12 X 30 IN. STRINGERS

forced concrete pedestal piers. This structure is of particular interest because of the extraordinary span lengths and dimensions of the timber stringers, the individual spans being 40 ft. long and the stringer system consisting of eight lines of 12 x 30 in. timbers.

As compared with the reinforced concrete viaduct, the composite type indicates economy for structures wherein the distance between grade and ground does not exceed 25 or 30 ft. and for crossings, of sloughs, swamps, overflow channels and the like.

An inspection of the tubular values given in section 5 will indicate that the maintenance expense for untreated timber trestles under Oregon conditions will vary between 60 cents and $2.00 per running foot per annum whereas the average annual maintenance cost for concrete bridges amounts to about 20 cents per running foot. Since the composite type has only been in service for a few years it is impossible to develop definite conclusions as to probable future annual maintenance expense. Cost data to date, however, indicate that the expense will be very little more than for concrete viaducts, the average cost to date being less than 25 cents per foot per year. It may be that over the years this will increase somewhat but present indications are that such expense will always be comparatively low. The reason for this doubtless lies in the fact that the substructure is protected from decay by means of a creosote pressure treatment; and furthermore, is protected from the ravages of the elements by the overhanging superstructure which acts as a waterproof shed or housing.

Discussion, to close in June 1939 JOURNAL, *should reach A. C. I. Secretary by April 1.*

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN JUNE 1939

Discussion of a paper by Conde B. McCullough

Maintenance and Repair of Bridges*

CONVENTION DISCUSSION

Duff A. Abrams‡: In looking at the photographs Mr. McCullough has shown, I wondered if he had investigated the matter of over-wet mixtures on the one hand and the curing of concrete on the other? That type of deterioration looks very much like excess water and inadequate curing, or both.

Mr. McCullough: All of these structures were built under engineering supervision and were supposed to have been of the proper consistency, properly cured. I say *supposed*, because many of these structures were built about 1919, '20, '21 and '22. As I recall, the Abrams theory was new in 1919. We were one of the first states to control our mixes by the water-cement ratio—I think we began it in 1920. We probably had 75 to 100 jobs going during those summers. Some of our resident engineers were pretty green and we were trying to give them a short course in cement technology. Maybe we got them dry and maybe we did not, but most of those later structures were well supervised; of those in 1919 and 1920, I am not so sure—there were some wet mixtures and sloppy methods of curing.

E. R. Bilnar‡: I happened to be, I believe, designing the bridges and walls of the Columbia River Highway in 1914. At that time we did not know of your law, Mr. Abrams, and I know that we had wet concrete, not sloppy—about four or five inch slump on an average. I will say that we did some curing and I am glad to say that I did not see any pictures from the Columbia River Highway reproduced here.

Mr. McCullough: That is correct; none of those terrible examples was in a bridge Mr. Bilnar designed.

*JOURNAL, Amer. Concrete Inst., Feb. 1939; *Proceedings* Vol. 35, p. 229.
†Consulting Engineer, N. Y. C.
‡Eddystone Portland Cement Co., N. Y. C.

T. F. Nichols:* In adding material for repair, would you give the chipped surface of concrete any particular treatment?

Mr. McCullough: We chipped it down; we were very careful to remove all the disintegrated material, probably going a little farther than necessary; we chipped it down until the reinforcement was exposed, and tried to clean it off as best we could. Then we applied a little coating of mortar, just a brush coat of mortar; we did not try any particular bond material. We made it reasonably dry and kept the surface moist, and so far our patching has turned out very nicely. Patching concrete is an old story with us. We have had some of our bridge crews in the field since 1919, and we still have with us five or six bridge and building foremen who have been with us for the full twenty year period. Some of those boys have a technique all their own and know much more about patching concrete than many engineers do. They have been able to make the patches stick.

C. G. Walker†: I understood the speaker to say he had found no relationship whatever as between the soundness test of the aggregate as measured by the sodium sulphate, and the freezing and thawing tests. I wonder if he has in mind what ranges in the sodium sulphate and the freezing and thawing losses he found in the aggregate.

Mr. McCullough: I think probably I did not make myself entirely clear. What I thought I said was this; in the case of those few structures wherein we found large pattern cracks (by far the minority), the aggregate used, on being re-sampled from its original source, showed up within the specifications on the sodium sulphate test. That was not true of all the structures you saw, just those few structures.

William N. Hudson‡: I should like to ask whether those pattern cracks showed up on the substructure in the case of the arch or abutment of piers as on the handrails?

Mr. McCullough: By and large, the most pronounced cracks occurred in the case of exposed horizontal surfaces, such as handrails, in the tops of handrail posts and exposed pier caps, although that is not one hundred per cent true. There was considerable evidence of cracking on some of the vertical surfaces, but generally the large pattern cracking, showed up on horizontal exposed surfaces. Another thing we have gotten out of these experiences is improvement in our design technique; we are trying wherever we possibly can to design

*Hornell, New York.
†Cast Stone Institute, Chicago.
‡William N. Hudson Co., Inc., Buffalo, N. Y.

our substructure with a rather pronounced superstructure overhang so that we will protect all of our substructures from the exposure of horizontal surfaces. We have given our pier caps a pronounced slope and our handrail caps a greater slope than before; we are trying to eliminate surfaces which will retain snow during the melting periods.

*A. B. Cohen:** Isn't there a tendency to overwork the surfaces during the finishing of handrails and caps?

Mr. McCullough: I think that is true, and I had a couple of micrographs to illustrate the point which I did not show. Long ago we discarded the practice of putting a mortar topping on handrail caps. We found out early in the game that mortar toppings would crack loose, because they have a different coefficient of expansion and of permeability. Some of the handrails shown were cast monolithic and floated just enough to get a finish. We believe that the floating of a handrail or a pier cap is something that can be overdone very easily.

Mr. Cohen: A great many of the old specifications for coping or handrail, added 20 or 30 per cent of cement: they needed more cement in the handrail or coping to get a better finish, and every one cracked.

*Consulting Engineer, N. Y. C.

Vol. 35 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN *FEBRUARY 1939*

"Ordinary Concrete"*

By Miles N. Clair†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

The author finds that "ordinary concrete," most of it made by common laborers, is a remarkable material in performing with "reasonable satisfaction" the functions intended. But he finds it costly, wasteful, cracked, spalled, honeycombed, leaky, non-uniform and unsightly. He tells *how*, by simple means, which he describes, and with modern knowledge, better specifications, and insistance that they be followed, good concrete can be had to justify, on a dollars and cents basis, the necessary additional supervision.

INTRODUCTION

It may be reasonably approximated that more than half of the construction volume in this country is of the type in which the cost of the concrete is not the major cost of the finished structure and where no special attention is given to the application of modern knowledge of how to get the best concrete. Such concrete is the "ordinary" concrete of this paper—in contrast with that represented by the remainder of the construction volume which includes generally, larger projects such as state highways, dams, bridges, buildings, and water structures normally constructed under supervision of special federal and state departments or by railroads and public utilities which have more fully applied the better concrete technique.

Ordinary concrete is a remarkable material in that although generally made by common laborers, without competent supervision, it has performed with reasonable satisfaction the functions for which it was intended. It is surprising that more of this concrete has not been unsatisfactory as to uniformity, strength, appearance, or durability.

*Received by the Institute Jan. 5, 1939 for presentation at the 35th Annual Convention, New York, March 1-3.
†Vice President, The Thompson & Lichtner Co., Inc., Boston, Mass.

It is a cause for wonder that architects, engineers, and contractors do not see the possibilities for improvement as to economy and quality.

Much has been done in the research laboratories in the last 25 years to develop a better understanding of the characteristics of concrete and the factors affecting them. On the relatively few large projects using hundreds of thousands of cubic yards of concrete this knowledge has been fully applied and even extended. Better and more economical concrete results. Application of such knowledge to the ordinary project has been slow because the architect and engineer have failed to demand it. Structures do not often collapse; much of the concrete is covered up by other material or it is painted. It is called "good enough" for the purposes intended. Cracks are accepted as inevitable along with the honeycomb, the spalls where honeycomb has been patched, the rust stains and the breaking out of the concrete over reinforcement, the leaking of concrete walls, the wide variation in strengths, and the non-uniform and unsightly appearance of supposedly architectural concrete surfaces.

The contractor who specializes in concrete construction and the material producers should be concerned about improvement. Their interest in the continuation and expansion of the market for concrete should make them foremost in promoting anything to improve it. There is little indication, however, of such interest (with a few notable exceptions) in the attitude of most of the contractors and producers toward the support of organizations promoting research and the distribution of information on good practice.

If the architect or engineer does not require the quality of concrete it is reasonably possible to obtain on the basis of our present knowledge it is either because they do not realize the possibility or because they do not know how to get it. Every architect and engineer cannot be a concrete expert, as concrete may represent only a very small percentage of the cost of the structures with which they deal. It is not necessary to become an expert to appreciate the possibilities of improvement by the application of available information. There is plenty of information in the easier-read technical literature indicating that something better can be done in structural design, in selecting materials, in concrete mixtures, and in construction procedure. If the data of Fig. 1 to 4 are typical of the average job, economy and better concrete will result from better control measures.

BETTER CONCRETE BEGINS WITH THE DESIGN

The interest of the architect and engineer in getting better concrete must start with his first consideration of it in the design and the speci-

fications. The design is frequently made under restrictions of antiquated building laws. They should be most active in obtaining revisions of such laws so that they may utilize more modern design methods such as those of the American Concrete Institute's Building Code or of the Joint Committee's 1937 Progress Report. These specifications encourage the use of more exact structural analysis for greater economy, for reduced cracking due to better positioning of reinforcing steel and for greater durability. There are large possibilities for economy in the use of modern concrete technique. The engineer should consider relatively higher design strengths so as to take advantage of the higher strengths safely obtainable with modern cements and concrete control. He will find it preferable to keep the cement contents at normal figures and obtain economy through higher design stresses than by using leaner mixtures, because of the inherent advantages of workability, relative freedom from segregation, and reduced effect of variations in aggregates on such mixtures as compared with lean mixtures.

As any designer must know, the only use of concrete on many small jobs is for sidewalk slabs, footings, and foundation walls where considerations other than structural strength frequently dictate minimum dimensions, allowing no structural economy. On larger projects the economy from high strength concrete will normally appear in beams, girders, columns, and footings due to reduction in dead load by the use of smaller sections.

The designer can help to improve ordinary concrete in still other ways than by better structural analysis and the use of strengths demanding good mixtures, if he will keep in mind always the fact that the structure he is designing must be built somehow by someone. The designer should not overlook the physical limitations imposed by conduits, openings, and other obstructions on the placement of both reinforcing and concrete.

BETTER SPECIFICATIONS FOR BETTER CONCRETE

The specifications written by the architect and engineer supplement the plans and describe the materials, equipment, and methods to be employed to obtain the quality of construction desired. To get better ordinary concrete, ordinary specifications must be better. Unfortunately no national engineering society or specification-writing body has yet produced a specification written with the purpose of serving the ordinary job.* Probably the most readily adaptable specification available is the Progress Report of the Joint Committee.†

*Except perhaps A. C. I. Tentative Construction Specifications for Concrete work on the Small Job, by Committee 506, *Proceedings* Vol. 27, p. 65 and 1184.

†Issued by Am. Concrete Inst., Jan. 1937.

The American Concrete Institute is giving the matter active consideration and much, of more value than can possibly be given in this short paper, will be found in reports being developed by Committees 611 Concrete Inspection, and 614 Recommended Practice in Measuring, Mixing and Placing Concrete.

The specifications in use on most ordinary jobs are scissors and paste copies of specifications originally written twenty or thirty years ago—ignoring current knowledge of good practice. Proportions are specified by arbitrary volumes such as 1:2:4. It would be an improvement at least to define the basis for these proportions. Originally they meant volumes damp and loose as measured on the job; then in many cases they came to mean dry and loose and now frequently are assumed to be dry and rodded volumes of aggregates. An architect or engineer who specifies 1:2:4 concrete in terms of damp and loose volumes, and gets 1:2:4 on a dry and rodded basis, is getting something very different from what he has in mind—since the difference in cement content might be 25 per cent.

It is important that the architect or engineer writing a specification for good concrete realizes that an arbitrary proportion specification does not determine the character of the concrete. The same proportions may be used with different materials producing differences in cement contents and other characteristics as great as the variation in interpretation of the volume basis previously given.

If the architect and engineer will accept the fact that there are reasonably definite laws to govern the relations of strength, water, cement, workability, durability, and permeability for given materials, under given conditions, and that constants necessary to fit into the equations representing these laws can be determined readily and at little cost, they will adopt a specification that makes it possible to get good concrete economically on the average job. Good concrete must be recognized to mean concrete that is uniform from batch to batch and from day to day so that in all parts of the structure the characteristics desired as to strength, durability, wear resistance, imperviousness, volume change, and appearance.

It may help again to state simply and briefly some of the more widely accepted facts in regard to concrete. The relation of strength to the ratio of the water to the cement content of any workable mixture follows definite laws. The constants vary for different cements and aggregates and can be determined by a few trial mixtures. The higher the water-cement ratio, the lower the strength of the concrete, all other factors being equal. High water content for a given cement content means a high water-cement ratio and low strength. A high

slump for a given cement content means a high water-cement ratio and low strength. Because large aggregates have less area per volume; they require less paste to coat them and less water for a given cement content. Low water-cement ratios give more durable concrete for given mixtures than high water-cement ratios. High water-cement ratios for given cement content mean increased water voids in the resulting concrete and increased volume change due to shrinkage. The more fluid mixtures with a given cement content have greater tendency toward segregation. All this sums up to the desirability of using concrete with as low a water content as is consistent with good workability. If this is accepted, then to specify good concrete as far as mixtures are concerned, all that is required is something as follows:

Class X concrete shall have a compressive strength of Y pounds per square inch at Z days and a water-cement ratio of not more than A gallons per sack of cement. The consistency as measured by the slump test shall be maintained within the limits of B to C inches. The coarse aggregates shall all pass an R inch size. The ratio of coarse to fine aggregates shall not be less than S to T.

In this type of specification all reference to compressive strength could be omitted if desired. The maximum water-cement ratio limit could be used alone or with the cement content to determine both compcompressive strength and durability. The limits to put in place of the letters A, B, C, R, S, and T will depend on experience. An excellent guide which needs no apology for repetition here is given in the Tables 1, 2, and 3 of the Progress Report of the Joint Committee, 1937.

Before these tables can be used properly as a guide for setting up the values in the specifications a decision must be made as to the minimum cement content needed to avoid trouble from poor workability. The equivalent of a 1:2.5:5* mixture or four and three-fourths bags of cement per cubic yard is the minimum cement content or leanest mixture that should be used on ordinary jobs and it should be used only for slabs on the ground and unreinforced and heavy sections. Mixtures for reinforced sections should contain not less than five and a half bags of cement per cubic yard. With these arbitrary limits (based on considerable experience) in mind, it can readily be seen from the tables that normal 2000 p.s.i. concrete can be expected from a mixture containing less than the 5.5 to 6 bags of cement required for the usual 1:2:4 mixture. Normally, better concrete in place will result if the structural design is based on the use of the 5.5 bag mixture and readily obtainable strength of 2500 p.s.i.

*Note that arbitrary mixtures on a dry and loose volume basis are used as reference throughout this paper because those who should be most interested in this paper think in such terms rather than in terms of the cement contents, water-cement ratios, mortar voids, and other relations used by concrete technicians.

TABLE 1—WATER CONTENTS SUITABLE FOR VARIOUS CONDITIONS OF EXPOSURE (GAL. PER SACK OF CEMENT)

	Severe or Moderate Climate, Wide Range of Temperatures, Rain and Long Freezing Spells or Frequent Freezing and Thawing					Mild Climate, Rain or Semi-Arid, Rarely Snow or Frost				
	Thin Secs.		Moderate Secs.		Heavy and Mass Sections	Thin Secs.		Moderate Secs.		Heavy and Mass Sections
	Reinf.	Plain	Reinf.	Plain		Reinf.	Plain	Reinf.	Plain	
A. At the waterline in hydraulic or waterfront structures or portions of such structures where complete saturation or intermittent saturation is possible, but not where the structure is continuously submerged:										
In sea water	5	5½		6		5	5½		6	
In fresh water	5½	6		6½		5½	6		6½	
B. Portions of Hydraulic or water-front structures some distance from the waterline, but subject to frequent wetting:										
By sea water	5½	6		6		5½	6½		7	
By fresh water	6	6½		6½		6	7		7½	
C. Ordinary exposed structures, buildings and portions of bridges not coming under above groups	6	6½		7		6	7		7½	
D. Complete continuous submergence:										
In sea water	6	6½		7		6	6½		7	
In fresh water	6½	7		7½		6½	7		7½	
E. Concrete deposited through water	*	*	5½		5½	*	*	5½		5½
F. Pavement slabs directly on ground:										
Wearing slabs	5½	6	*		*	6	6½	*		*
Base slabs	6½	7	*		*	7	7½	*		*

G. Special Cases:

(a) For concrete exposed to strong sulfate ground waters, or other corrosive liquids or salts, the maximum water content should not exceed 5 gal. per sack. See Sec. 601.

(b) For concrete not exposed to the weather, such as the interior of buildings and portions of structures entirely below ground, no exposure hazard is involved and the water content should be selected on the basis of the strength and workability requirements.

*These sections not practicable for the purpose indicated.

TABLE 2—CONCRETE OF MEDIUM CONSISTENCY (4-IN. SLUMP)*

Max. Size Coarse Agg. in.	Estimated 28-Day Comp. Str. p.s.i.	Cement Factor Sacks Cement per cu. yd. Freshly Mixed Concrete	Max. Water per Sack Cement Gal.	Fine Agg. % Total Agg. by Weight (Range)	Approx. Wts. Saturated Surface—Dry Agg. per Sack of Cement—lb. (See Note B below)		
					Total	Fine Agg.	Coarse Agg.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	((8)
1	2250	4.9	8	40—46	660	280	380
2	2250	4.5	8	37—43	740	300	440
3	2250	4.1	8	34—40	840	310	530
1	2750	5.6	7	39—45	570	240	330
2	2750	5.1	7	36—42	640	250	390
3	2750	4.7	7	33—39	720	260	460
1	3000	6.0	6½	38—44	520	210	310
2	3000	5.5	6½	36—42	590	230	360
3	3000	5.1	6½	34—40	660	240	420
1	3300	6.5	6	37—43	470	190	280
2	3300	6.0	6	35—41	530	200	330
3	3300	5.5	6	33—39	600	120	380
1	3700	7.2	5½	36—42	420	160	260
2	3700	6.7	5½	34—40	470	170	300
3	3700	6.2	5½	32—38	530	180	350
1	4250	8.0	5	35—41	370	140	230
2	4250	7.4	5	33—39	420	150	270
3	4250	6.8	5	31—37	470	160	310

NOTE B—The approximate weights in columns (6), (7), and (8) are based on a bulk specific gravity of 2.65 in a saturated surface dry condition.

*Since the quantities in Table 2 are all approximate, it will be sufficiently accurate to modify the cement content in accordance with the following: For each 1-inch difference in slump, change the cement factor ¼ sack per cu. yd., increasing this factor for slumps greater than 4 in. and reducing it for slumps less than 4 in.

TABLE 3

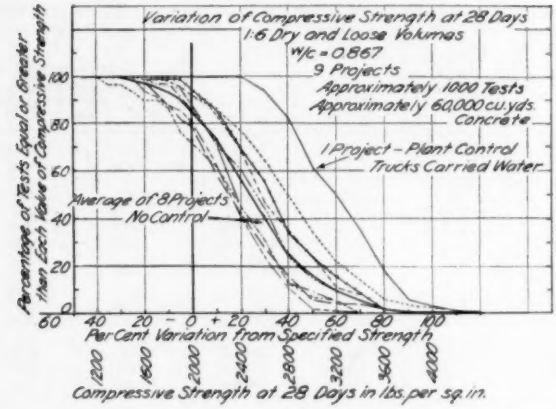
Portion of Structure	Consistency—Slump		Maximum Size of Coarse Aggregate inches
	Maximum in.	Minimum in.	
Reinforced foundation walls and footings	5	2	1½
Plain footings, caissons and substructure walls	4	1	2
Slabs, beams, and reinforced walls	6	3	1
Building columns	6	3	1
Pavements	3	2	2
Heavy mass construction	3	1	3 to 6*

*In making the slump test, all aggregates larger than 2 inches should be screened out of the mixture.

The mixtures given in Table 2 may be used as an indication of what might be expected but should not be specified. The actual mixtures should be determined by trial for the job materials and this should be covered by a paragraph similar to the following:

The proportions of materials to give most economically the desired consistency and strength within the limiting water-cement ratio (or cement content) shall be determined previous to the start of the work by a competent concrete laboratory selected by the architect or Engineer and paid by him.

This paragraph introduces at least three controversial questions. All concrete laboratories and concrete technicians are not necessarily competent, as there is required a knowledge of both construction and


Variation of Compressive Strength at 28 Days
1:6 Dry and Loose Volumes
W/c = 0.867
9 Projects
Approximately 1000 Tests
Approximately 60,000 cu. yds. Concrete
1 Project - Plant Control
Trucks Carried Water
Average of 8 Projects
No Control
Percentage of Tests Equal or Greater Than Each Value of Compressive Strength
Per Cent Variation from Specified Strength
Compressive Strength at 28 Days in lbs. per sq. in.

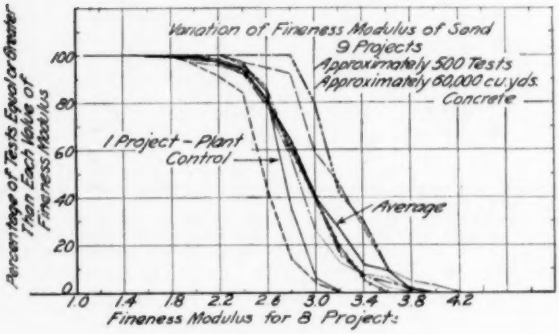

Variation of Fineness Modulus of Sand
9 Projects
Approximately 500 Tests
Approximately 60,000 cu. yds. Concrete
1 Project - Plant Control
Average
Percentage of Tests Equal or Greater Than Each Value of Fineness Modulus
Fineness Modulus for 8 Projects

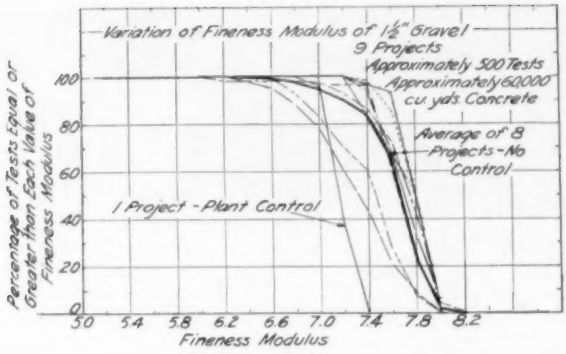
FIG. 1 (TOP) FIG. 2 (BOTTOM)

concrete technology in order to give proper advice on mixtures that will work in the field and such knowledge is not obtained over night and is not necessarily obtainable for the lowest fee. The selection of the concrete laboratory by the architect or engineer and payment by him is desirable to avoid making the contractor responsible for an engineering or architectural function. Whether the cost should come out of the engineer's or architect's fee or be a separate charge to the owner is a matter for the professional societies to answer.

The characteristics of the cement and aggregates, as has been mentioned previously, decidedly affect the characteristics of the concrete. The effect on strength and consistency or workability will be determined by the tests specified.

The choice of the cement

Quality of cement as reflected by composition should be guarded by a standard paragraph. If reduction in volume change and improve-



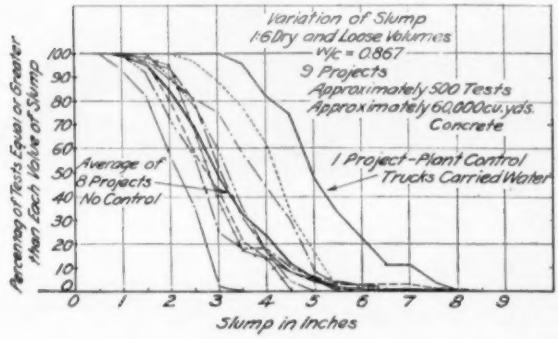


FIG. 3 (TOP) FIG. 4 (BOTTOM)

ment in durability are of special importance, the specification should refer to Federal Specifications, SS-C-206, Moderate Heat of Hardening Portland Cement, or the specification for such cements which will probably eventually come from Committee C-1 of the American Society for Testing Materials. Such cement is normally darker in color than regular portland cement and somewhat slower in strength gain. Modifications of portland cements and the use of admixtures should not be permitted unless full data are available on their effect on all the important characteristics of concrete and then only on the advice of a competent concrete consultant.

The Aggregates

The importance of the quality of the aggregates has been lost sight of to a considerable extent in recent years. Yet the characteristics of the aggregates have a major effect on the concrete. They must be sound, clean, and well-graded. When good gradation is hard to obtain (and that may be often) economy is affected adversely because workability must then be obtained by higher cement content. Keeping the

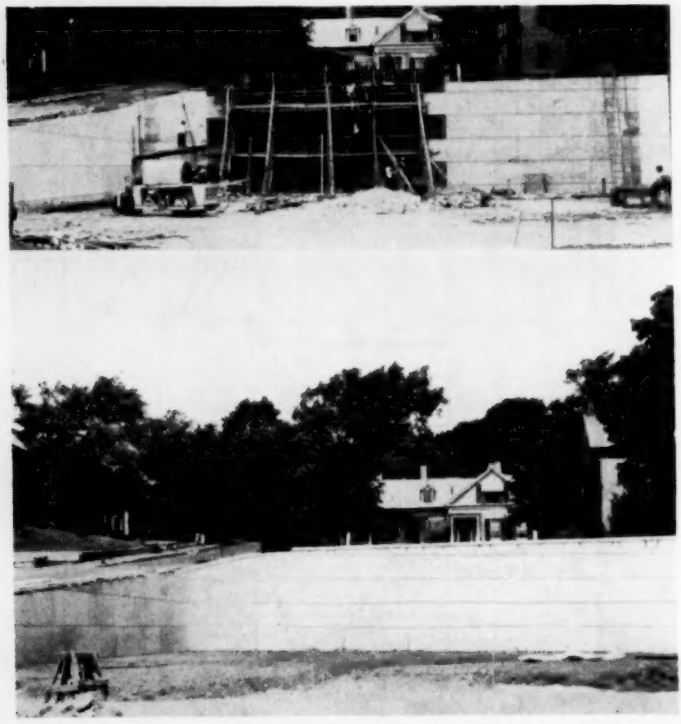


FIG. 5—AN ORDINARY CONCRETE JOB WELL DONE, THE ARCHITECT USING METHODS SUGGESTED IN THIS PAPER WITHOUT A CONTROL ENGINEER

FIG. 6—CONDITIONS REQUIRING ATTENTION TO AVOID POOR FOOTINGS

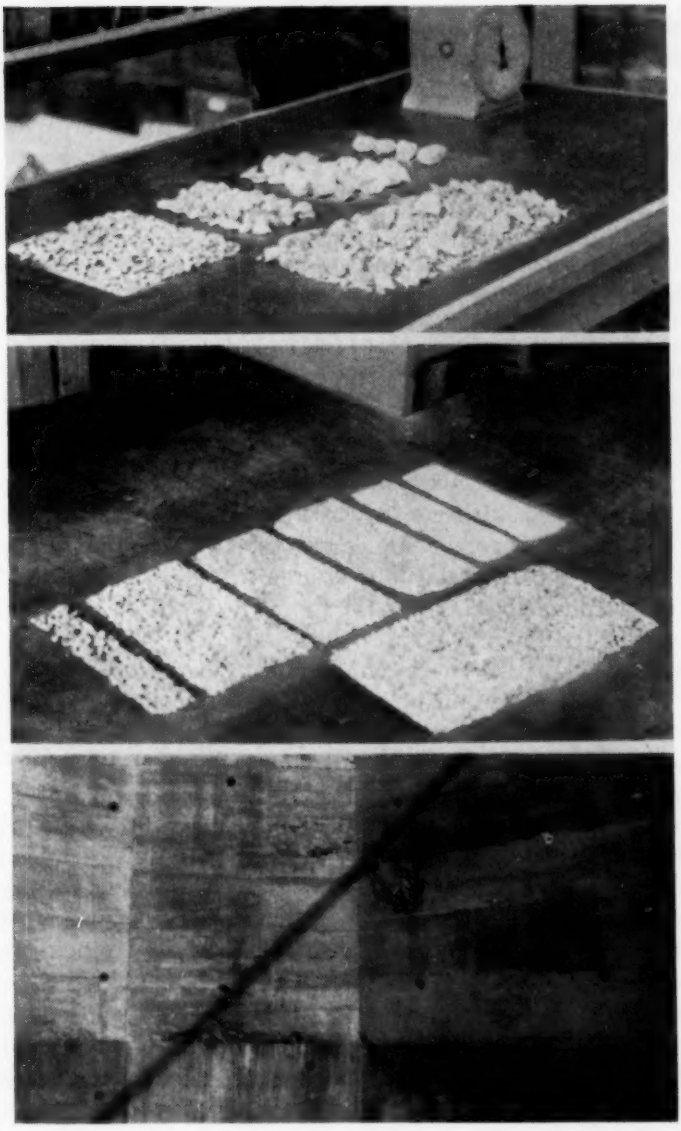


FIG. 7—(TWO UPPER VIEWS) THE TESTING LABORATORY CAN HELP THE ORDINARY JOB WHICH DOES NOT HAVE INSPECTION BY FURNISHING SAMPLES SUCH AS THESE FOR COMPARISON

FIG. 8—(BOTTOM) WHAT MIGHT OTHERWISE HAVE BEEN A GOOD JOB WAS SPOILED BY TRYING TO PLACE CONCRETE WHICH THE FOREMAN KNEW WAS DEFICIENT IN SAND AND TOO DRY AND FAILURE TO USE GROUT AT EACH "LIFT"

gradation uniform, irrespective of the gradation easily obtainable, is of great importance in getting uniform concrete. An idea of how the gradation varies on ordinary jobs is given by Fig. 2 and 3. A specification for ordinary concrete which will help to reduce variations in gradation if enforced might be as follows:

Concrete aggregates shall meet the requirements of the Tentative Specifications for Concrete Aggregates of the American Society for Testing Materials C33-37T with the addition that the fineness modulus during the course of the work shall not vary more than 0.1 from the fineness modulus as determined on the approved samples. Complete tests of aggregates to determine the properties defined by the specifications shall be made previous to the start of the work by a competent concrete laboratory selected by the Architect or Engineer and paid for by him. The Concrete Laboratory shall furnish a 25-lb. duplicate of the sample of aggregate tested and accepted by it for use as reference at the site.

Construction specifications

With materials and mixtures specified the architect and engineer are concerned next with construction specifications. Good concrete will not result unless good equipment is used with good methods. The 1937 Progress Report of the Joint Committee and the A. C. I. Tentative Construction Specification for Concrete Work on the Small Job, give good instructions in detail on most items of importance. They may well be adopted. They would help to warn the contractor specifically in regard to equipment before he gets the contract rather than after he has it on the job. A paragraph such as follows has been of assistance:

The Bidder is informed that the requirements of this specification as to quality and uniformity of concrete will be strictly enforced and that methods and equipment which do not allow the maintenance of such quality will not be tolerated.

The design having been made with the recognition of the problems of construction and the specifications prepared with the purpose of making it possible to get and require good concrete, the next step and the most important of all is the actual construction. What finally is wanted is good concrete in place, not perfect concrete on the drawings, in the specifications, or in the concrete laboratory. The architect or engineer can help himself here by careful selection of the contractors allowed to bid on the job. The ideal is of course to select the best contractor and give him the job but the ideal does not often prevail in regard to the ordinary concrete project. In the consideration of contractors the quality of the concrete construction done by a contractor in the past is the best evidence of the quality of the work he will perform. It is well before the start of the work to discuss the matter of concrete with the contractor so that he understands thoroughly what is wanted and has no alibis. If this seems to be unduly critical of the contractors

on average jobs, remember again that on most such jobs concrete represents only a relatively small part of the total cost. The job is normally let on a purely competitive basis withc a multitude of bidders and will frequently be let to a contractor with relatively little, if any, permanent concrete organization.

SUPERVISION

Supervision of the job by the architect or engineer is necessary to get better concrete from the average contractor. This may be done personally on small jobs, by a clerk-of-works on larger jobs, and with the assistance of outside concrete laboratories on still larger projects.

Excavations

Whoever does the supervision or inspection must see that excavations are properly prepared before concrete is placed in them so that the concrete will not be contaminated by falling soil, placed on frozen ground, or in water.

Forms

Forms must be examined for tightness, stability—ability to carry the loads, surface finish and treatment to prevent sticking, trueness to lines and dimensions, and freedom from debris. The forms must be arranged so that it is possible to get concrete into place without unrestricted dropping of more than a few feet or flowing over distances of more than ten feet. It must also be possible to introduce into the forms equipment for compacting the concrete after it is in place. Leaky forms cause much of the disfiguration of concrete and too many re-uses of forms result in open joints and leakage.

Reinforcing

Reinforcing steel should be inspected at the source to see that it is of the quality specified. The principal danger as far as quality is concerned is from getting steel of lower strength than specified or so brittle that it can be cracked in placement without detection. The more important matters are to see that the steel is clean, that it is of the proper size and form, that it be positioned properly and so held that placement of the concrete will not displace it. A great deal of the spalling of concrete over reinforcement is due to improper placement of steel or its displacement during placement of concrete. Experience indicates that a careful check on the reinforcement left ready for the concrete placement by the steel men on almost any job will disclose a major error either in position or amount of reinforcement. A good deal of the trouble with reinforcement would be eliminated if attention were given to scheduling operations so that there is ample time to get the steel ready and checked before concrete is placed.

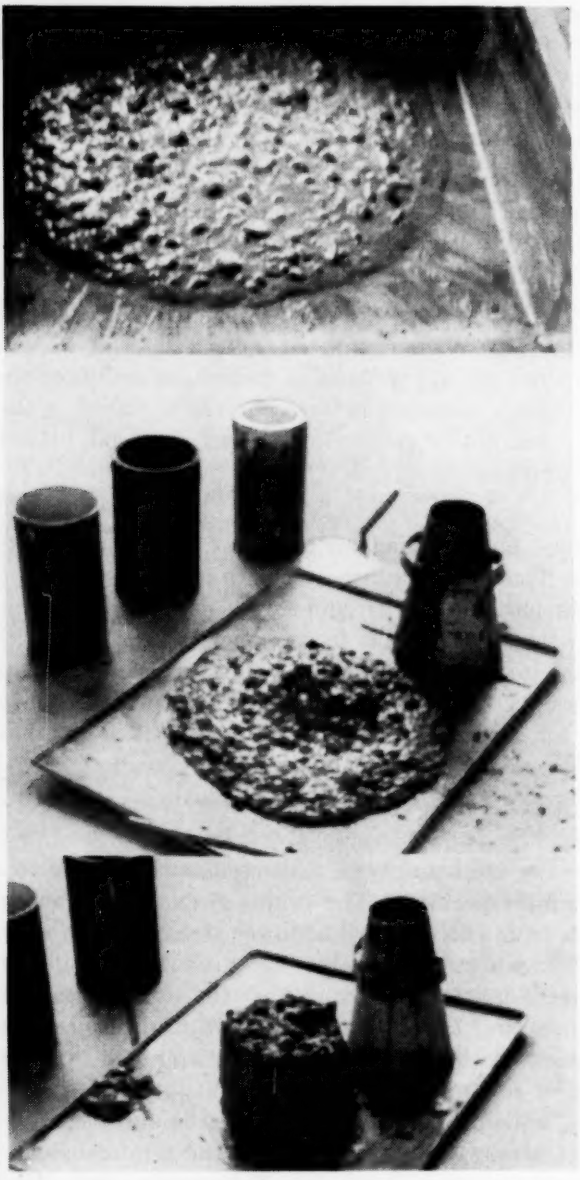


FIG. 9—JUDGING THE CONSISTENCY: (TOP)—CONCRETE NEVER NEED BE MORE FLUID THAN THIS; (CENTER) TOO FLUID EXCEPT FOR THIN OR HEAVILY REINFORCED SECTIONS; (BOTTOM) A GOOD PLASTIC MIXTURE SUITABLE FOR MOST CONCRETE

Proportioning and mixing

The proportioning and mixing of concrete is either performed at the site or at a ready-mixed concrete plant. If done at the site, the measurement is in most instances by volume. Though possible, it is extremely difficult to get uniform concrete where volume measurement is used. Scales for measuring wheelbarrow loads are available at low cost. If volume measurement is used, the architect or engineer should get information from the concrete laboratory as to the volume corrections for moisture bulking and mark the measuring hopper and wheelbarrows. The volumes of the coarse and fine aggregates to be used will have been determined by the concrete laboratory at the time of making tests of the aggregates. Determinations of moisture content should be made at intervals to give corrections for bulking and water content. After some experience with the given material the moisture contents can be estimated and determinations need be made once or twice during a day's work. If such determinations cannot be made, close observation of the workability will indicate needed changes in fine and coarse aggregates and water content to keep workability and consistency constant. With an observed increase in water content, proportions may be kept constant by increase of aggregate and decreased water. If the coarse aggregate, as is usual, remains practically constant in water content and the sand increases in water content, naturally the apparent sand content must be increased relative to the coarse aggregate. Where weight measurement is used, the correction for moisture content is more definite and easily made as a correction in weights depends on actual or estimated moisture contents. It must not be overlooked that unless reasonably accurate corrections are made for moisture, not only will the mixture not be correct as to cement or water-cement ratio but the yield also will be in error.

If aggregates vary in gradation, the workability of the concrete will change correspondingly and particularly where volume measurement is used. Some adjustment can be made to allow for changes in gradation so as to keep workability reasonably constant but the variation of aggregates on many ordinary jobs requires the full-time attention of a concrete technician. There is no real reason why aggregates should vary so greatly. Reasonable care in processing, storing, and handling will enable any up-to-date aggregate producer to furnish aggregates within the 0.1 fineness modulus tolerance suggested above. If the aggregates are within this tolerance, a change of a maximum of 100 lb. per cubic yard of concrete up or down of the fine and correspondingly down or up of the coarse aggregate will give correction for

practically any change likely to occur except a change in amount of crushed particles.

Aggregates may be damaged on the job and the architect or engineer should be particularly careful to forbid storage on the ground where contamination can occur or storage in stockpiles of such size and shape that appreciable segregation will occur. Flat, rather than conical piles, and separation of coarse aggregates into several sizes reduce segregation.

Mixers have definite capacities specified by the manufacturers. If used in excess of these capacities, the mixing within normal mixing time will not be satisfactory. Some visual or audible timing device should be used so that a full-minute mixing (for mixers up to one cubic-yard capacity) time will be obtained. Mixers must be kept clean and blades kept up to size. Any mixer that discharges one consistency and mixture at the start and another at the end of the load is not functioning correctly when the concrete is of less than 6 in. slump. Such mixers must be taken off the job immediately if good concrete is to be obtained.

It is axiomatic that the water content should be kept definitely under control at all times. The question is how! Equipment is available for continuous determination of moisture content but none is particularly adapted to the average job. One method has been indicated in a previous paragraph. The best solution for most average jobs is to have the proportioning and mixing under the direct supervision of a concrete technician who can determine gradations and moisture contents of the aggregates and adjust the quantities to correct variations. To do this there must be some accurate means of measuring water. The water tank on most job mixers apparently can't be kept in condition to measure accurately because of dirt in the water, leaks, change of level, and vibration. A simple calibrated tank at the mixer is a good water measuring device.

The variation in moisture content of aggregates will be greatly reduced if they come from stockpiles. It is nearly impossible to control water if aggregates come direct from washing plants. If the aggregates are kept constant as to gradation and moisture is properly allowed for, the consistency and workability will remain constant. Variations in consistency or workability give a quick visual indication of variation in mixture.

Ready-mixed concrete

If ready-mixed concrete is used, the concrete comes to the site ready for use. In many cases this would appear to be the ideal arrangement because a ready-mixed plant with its large total output *should* be

able to have the superior batching and mixing equipment and the best of technical control. Unfortunately this is not the case. Some of the ready-mixed plants are actually no better than job volume-measurement plants although most have weighing equipment for aggregates and cement and volume measurement for water. Few have even reasonably adequate tchnical supervision.

There are two general types of ready-mixed concrete operations of interest to the average job, (1) the central-mixing plant where the concrete is mixed in a regular mixer and then shipped to the job in agitator trucks and (2) the central-proportioning plant where the materials are batched into a truck mixer and mixed en route or at the job. In either case the concrete should be placed within an hour after being put into the transporting vehicle, but temperature and other conditions should be considered as requiring variations in this limit. The central plant mixing operation allows superior control of the concrete. What the concrete looks like can be seen before it leaves the plant and corrections can be made. Truck-mixed concrete cannot be seen until it is ready to discharge and then unless carefully supervised, the attempt is made to correct all defects by adding water. It will help to improve the quality of average concrete supplied from ready-mixed concrete plants if it be required that it comply with the Standard Specifications for Ready-Mixed Concrete, ASTM C94-35, and that each load be covered by a certificate from a competent concrete laboratory.

Placing concrete

Concrete on the average job should be placed by buggy or drop-bottom bucket and not by chutes because of the impossibility of control of concrete chuted directly into place and the common difficulty of getting contractors to arrange the chutes so that they will be of proper slope to carry good concrete. Chutes may be used to distribute concrete to properly designed distribution hoppers which feed buggies or drop-bottom buckets. Distribution hoppers must have adequate gates so that the specified concrete will flow through them. Concrete should be placed in approximately horizontal planes to limit flow and segregation. Failure in this causes the unsightly (and bad from a quality standpoint) sloping planes seen on most ordinary concrete. It should be deposited as near its final position as possible so as to avoid segregation.

At the bottom of deep forms such as columns, walls, and girders where there is much steel and particularly where jointing with previously placed concrete, a layer of neat cement grout an inch thick or several inches of 1:1 mortar should be placed to give good bond and

avoid honeycomb. There should normally be a man spading the concrete in the forms for each bag capacity of the mixer. If vibrators are used, they must be handled by experienced men and kept in motion to avoid segregation of the concrete. A good vibrator properly handled will replace two or three men.

After the concrete is placed, proper curing will add as much to its quality as the care taken in proportioning and placing. During normal or warm temperatures concrete should be kept moist for at least seven days. This is usually not done unless the contractor is frequently cautioned. As the important thing is not to lose water, covering with wet sand or waterproof paper or keeping on the forms are good ways to keep the moisture in the concrete. In cold weather not only should the concrete be placed with a temperature of at least 50°F. but it should be maintained at 70°F. for at least 24 hours to avoid damage by freezing. Greater protection is necessary, of course, if normal rate of strength gain is desired. Enclosure of the concrete and the use of wet steam as the heating medium gives the best protection. Unless there are facilities at the job for heating water and aggregates and the protection of the concrete, concreting should not be allowed on the ordinary job when the temperatures go below 40°F. When there are proper facilities, concreting can be done with temperatures down to 20°F. without particular difficulty. At temperatures below this, the procedure to be followed should be submitted to a concrete consultant for approval.

Stripping

The age of concrete when forms may be stripped depends on the strength, quality, curing conditions, and loads being carried. The important thing is to avoid damage to the concrete in stripping. This means first that forms must have been designed properly and second that the stripping gang should avoid the procedure of a wrecking crew. If the spirit of what already has been said is followed, the concrete has been carefully controlled and placed and there will be very few voids to patch. Those few should be repaired immediately by cutting back to sound material, coating the surface with thick neat cement grout, and filling to proper surface with mortar or concrete mixture. This work is best done by a good concrete finisher.

CONCLUSION

The ordinary job may range from a few yards to many thousands and yet all that is given here and more can be done to improve the quality of the concrete without greatly increasing the cost. On the small job the tests of all the materials and the services of a control

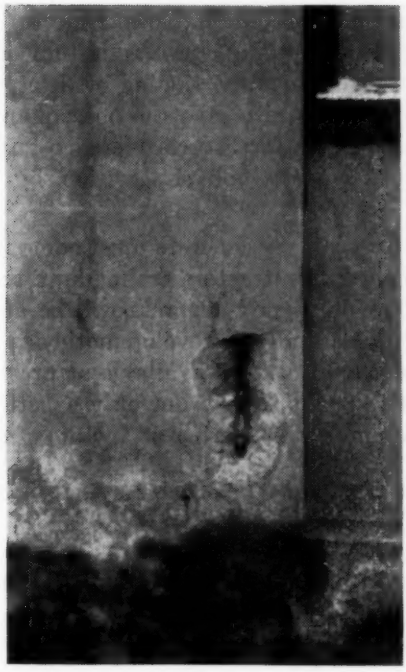


FIG. 10—WHAT HAPPENS WHEN STEEL IS TOO NEAR THE SURFACE

engineer for the proportioning would not cost more than fifty cents per cubic yard. The architect or engineer would of course have to give more attention to the concrete placement and curing but that is his duty to the owner. On larger jobs the services for testing and control would not cost more than thirty cents and might be as low as twenty cents per cubic yard. Supervision of the placement could be obtained for another fifteen cents per cubic yard.

If we assume that even these sums, so small compared with the whole cost, are not to be spent, still decided improvement in quality can be obtained by visually checking aggregates against a satisfactory sample, checking mixing time, and consistency, persistently keeping all concrete that does not look satisfactory out of the forms, (What does the cost of a few rejected buggies of concrete amount to, compared with the damage they do and the cost of patching!) and carefully supervising the placement and curing. Specimens may be taken from concrete as placed for testing in order to check the compressive strength. It is recognized that by the time the tests are obtained they are simply history. Many ordinary jobs do not progress so rapidly

but that the test results are obtainable in time to be of value. The tests give a warning if something goes wrong that cannot be detected visually such as an error in proportioning or deleterious materials in the aggregates. Such tests also give the only quantitative record of the quality of the concrete for future reference and are particularly valuable when there is little job supervision by the architect or engineer.

The urge for better ordinary concrete must come from the architect or engineer and when the contractors are made to realize by rejection of poor concrete that only good concrete will be accepted, they will demand better materials and will develop methods and equipment far simpler and more economical than available at present to obtain this better concrete. The result will be not only a better concrete but a more economical concrete, considering all factors of cost.

Discussion to close in June 1939 JOURNAL, *should reach A. C. I. Secretary by April 1.*

JOURNAL
of the
AMERICAN CONCRETE
INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN JUNE 1939

Discussion of a paper by Miles N. Clair

"Ordinary Concrete"*

CONVENTION DISCUSSION

H. F. Thomson†: It is unfortunate that the people who ought to hear this kind of paper are those not in the audience; I think Mr. Clair will agree. I wish to comment on two points, although there are many features of the paper which justify consideration by all of us. The paper points out the lack of supervision on the part of many architects. Those of us who have made an honest attempt to offer good concrete to the trade—better than the trade is willing to pay for—are constantly up against a lack of insistance by the architect that he get on his hob the kind of material he thinks should go in. It is really pathetic to see the number of architects and engineers who will write into their specifications "one-two-four concrete," and assume that from that point on they are going to get proper concrete into their buildings. In attempting to offer a reasonably well supervised concrete in competition with haphazard job-mixed concrete which results from the manifestly lax attitude of architects toward insisting upon good material, the general situation is exceedingly discouraging, and it would be constructive if the major points Mr. Clair has pointed out could be brought home to the average architect and engineer.

Another point made in the paper is in reference to supervision by laboratories. I wish that our territory had the benefit of the type of supervision that Mr. Clair's organization is in a position to furnish. The continued sad experience of having so-called well constituted laboratories fall down on their job after undertaking to give control service, is a very serious matter. Recently one of the national laboratories reported cylinders as testing 275 lb. The laboratory was

*JOURNAL, Amer. Concrete Inst., Feb. 1939; *Proceedings* Vol. 35, p. 257.
†General Material Co., St. Louis, Mo.

responsible for making, curing and testing the cylinders, and it did not even point out to the architect that the cylinders had been frozen. Ultimately, after severe criticism of the people who furnished the concrete, it was necessary to take out a portion of the job and replace it, and the concrete in place tested well over 2500 lb. That kind of failure of responsibility on the part of a laboratory is just short of criminal. There again the man who ought to receive the suggestions Mr. Clair is offering is not in the audience. But I take this opportunity of pointing out that those two conditions of lack of proper supervision on the part of the architect and the apparent carelessness or inability on the part of the laboratories to perform as they are paid to perform, are two very serious factors in connection with obtaining a good concrete in our jobs.

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *FEBRUARY 1939*

Mixtures, Placing and Curing for Architectural Concrete*

By R. S. Phillips†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

Architectural concrete must be sightly—not only at the completion of the job but thereafter. To get satisfactory concrete for exposed walls is a matter of quality, uniformity and durability. It is not possible to produce architectural concrete by the procedures common to structural concrete—especially that structural concrete which has sufficed for protected structures. Architectural concrete means "taking pains"—in form work, materials, mixtures, placing, curing; applying new skill and new technique to meet a more exacting demand.

Widespread attempts to adapt to architectural concrete, specifications and construction practices developed in the field of structural concrete have resulted in some disappointments to owners and architects. A fuller understanding of the quality and workability of concrete required to insure an enduring structure of pleasing appearance will result in more complete specifications and more intelligent bids.

Structural concrete is primarily utilitarian. Since, in building construction, it has usually been concealed, appearance has been deemed of little importance, and concrete specifications of a somewhat general nature, have, therefore, become common practice. The protection from weathering afforded by the outside facing material and the design requirements have served to focus the attention of architects and engineers on compressive strength rather than on durability. Specifications for concrete usually included fixed proportions of cement, sand and coarse aggregate. The aggregate specifications were frequently loosely worded with few requirements as to grading and were often indifferently enforced. Specifications for control of the quantity of

*Received by the Institute January 4, 1939.
†Portland Cement Association, Chicago.

mixing water, if any, were limited to a few generalizations concerning consistency or slump of the mix. Curing of concrete during the early hardening period, even if mentioned in the specifications, was more often neglected than practiced.

The improvements in concrete technique, which have characterized concrete highway and hydraulic construction during the last ten years, have been slow to find their way into building construction. In those sections of the country where architectural concrete has become commonplace, progress in this field has far outstripped that made in other sections.

The first principle which the architect and engineer must accept is that concrete which is to be exposed to weathering, principally the action of water and freezing, must be more durable than that which is protected from the elements. The second is that a much more workable mixture is required, and the concrete must be placed with greater care, to eliminate the common defects which have been tolerated in structural concrete.

The resistance of concrete to weathering, its durability, is subject to the same control as its compressive strength. Assuming that the aggregates are composed of durable particles, the factors which improve the compressive strength also improve the durability. It is possible, however, for concrete to be adequately strong to satisfy the structural requirements and yet not be sufficiently durable for the climatic conditions to which it is exposed.

USE LESS WATER

The effect of the water-cement ratio on the compressive strength of concrete has been recognized for many years. Its effect on the durability, while not as generally appreciated, is equally well established. A difficulty in securing adequate specifications has been the assumption on the part of many that control of the quantity of mixing water is synonymous with the use of dry mixtures. Unfortunately also in many cases where the maximum permissible quantity of water has been specified, little attention has been given to enforcement of this clause. The mistaken impression is that careful control of the quality of concrete in building construction is impracticable, and that haphazard methods are unavoidable.

Specifications for architectural concrete should definitely limit the quantity of water to be used and the requirement should be rigidly enforced. In most cases the water should not exceed 6½ gallons per sack of cement including the free moisture carried by the aggregates. If proportions are specified, they should be such that the mix

can be made easily placeable without exceeding the maximum water content permissible. Provision should then be made for adjusting the relative quantities of sand and coarse aggregate where necessary to obtain the required workability. This is essential to secure a cohering, plastic mix. If in the stated proportions, the sand is equal to 40 per cent and the coarse aggregate 60 per cent of the total aggregate on the basis of dry volumes, the amount of adjustment necessary will be small. This calls for a larger proportion of fine material than is generally required in structural concrete.

LARGER AGGREGATE

Instead of adhering to the practice of specifying coarse aggregate of ¾- or 1-in. maximum size, this limit should be increased to 1½ in. This will result in more workable mixtures and the use of less water in mixtures of equal cement content. With a properly balanced mix, and with reasonable care in placing, there is less danger of honeycomb or rock pockets when using a well graded 1½-in. coarse aggregate than with a ¾-in. maximum size. Complete coverage of the reinforcing steel is easily secured even with the larger aggregate. The particle sizes should be well distributed from ¼-in. to the maximum. Control of grading on the job will be greatly facilitated if the coarse aggregate is delivered in two separate sizes, ¼ to ¾-in. and ¾ to 1½-in., and these sizes measured separately into each batch. Many architects are now incorporating such a requirement in their specifications and it is highly recommended.

Control of the sand grading is equally important. Sands which are deficient in fine particles and those which contain excessive quantities of any one or two particle sizes are particularly objectionable. The use of such sands permits excessive bleeding of the mix with consequent water gain and sand streaking. Specifications should limit the percentage passing the No. 50 sieve to a minimum of 15 and a maximum of 30. The percentage passing the No. 100 sieve should be at least 3 and not more than 8. The percentage passing any one sieve and retained on the next smaller sieve of the standard series should be limited to 40.

UNIFORMITY OF MIX

Uniformity of the concrete mix from batch to batch and from day to day is essential. This applies not only to the consistency, but also to the proportions entering the batch. The possible error in volumetric measurement of sand has long been recognized, but so far as building construction is concerned, little has been done about it. Weighing of aggregates has been required on large engineering construction for ten years or more. Meanwhile, equipment has been developed which

renders weighing not only feasible but profitable on any concrete job from the half-sack mixer project to the eight, four-yard mixer plant at Grand Coulee Dam. With the most accurate volumetric measurement, a change of as little as one per cent in moisture content of the sand may result in an error of ten per cent in the actual quantity of dry sand entering the batch. Where weighing is practiced, the corresponding error in measurement will be only one per cent.

Obviously, accurate measurement of the mixing water is essential. Modern mixers are equipped with reliable water measuring devices, but it is wise to calibrate them before starting the job. Suitable tanks or meters are obtainable for attachment to mixers not so equipped. Expensive or complicated gadgets are not necessary, since an improvised measuring device, properly used, will produce satisfactory results. The moisture carried by the aggregates, principally the sand, will normally amount to from 1 to 2 gallons per sack of cement. If the sand is stockpiled and allowed to drain for at least 24 hours before use, the variation in moisture content will not exceed one or two per cent. Each per cent change will equal approximately 1 quart per sack and will change the slump of the concrete by about 1½ inches. Such a change can be recognized quickly, and the quantity of mixing water changed to restore the original consistency. While an experienced eye can detect a change of as little as one inch in slump, it is advisable to make slump tests occasionally as a check. So long as the aggregate proportions are measured accurately and with reasonable uniformity of aggregate grading, the only thing which will cause a change in the slump is a change in the moisture in the aggregates. Since this is easily observed, frequent moisture tests of the aggregates are not necessary. Such tests should be made occasionally, however, to make sure that the total mixing water is within the specified limit.

MIXING TIME

More attention should be given to the mixing time than has been customary on building construction heretofore. To this end, a timing device with a signal bell is desirable. The time of mixing, measured from the time when all materials have entered the mixer to the time when the first concrete is discharged, should be at least one minute for batches of ½ cu. yd. or smaller. For a ¾ or 1-cu. yd. batch, this should be increased to 1½ minutes. Excessive mixing or standing for extended periods after mixing stiffens the mix. The addition of water to retemper affects the quality the same as if this water had been part of the original batch. So long as the concrete is sufficiently plastic so that it can be placed without the addition of water, the quality has not been impaired by the delay.

CONVEYING AND PLACING

How closely the quality and uniformity of the concrete, in its final position in the forms, will approach that with which it leaves the mixer, depends on the care taken to prevent segregation during transporting and placing. Even a high quality concrete can be so abused that the concrete in place is far from satisfactory. Since water is the lightest ingredient of the mix, it is displaced by the heavier solids as they tend to settle, thus causing what is commonly known as water gain or bleeding. Since the cement tends to move in the opposite direction to the water, the final result is an increase in the water-cement ratio toward the top of each lift. Such a separation takes place to a greater or lesser extent in transit, frequently producing a layer of water or thin paste on the top of the buggy. When the buggy is dumped, this liquid splashes into the forms ahead of the stiffer concrete, while the latter frequently must be scraped out. This tendency to separate in transit is greatly reduced by using well graded sand and a balanced mix, and by avoiding overwet consistencies.

Water gain and bleeding in the forms may seriously reduce the durability of the concrete in certain locations and is very likely to mar the appearance of the finished structure. Failure to absorb or dispose of accumulated water at the tops of walls and in the zones immediately below construction joints results in porous, non-durable layers at these locations. This produces a dark band which stands out in contrast with the lighter color of the denser concrete, even after the surface is uniformly dry. At the top of such a wall the effects of freezing and thawing will be apparent in a very few years, and serious disintegration may follow. Below a construction joint, where weathering is less severe, similar damage may be delayed for many years, but there is always the threat of trouble should water find its way into the joint. These undesirable conditions should be avoided by stiffening the mix (reducing the mixing water) to absorb the excess water as the top of the lift is reached. If there is any indication of "soup" at the top of a lift, the contractor should be required to remove the overwet concrete from the forms.

Before starting any lift, whether at the top of a footing or at a construction joint in the wall, the milky scum which coats the surface should be removed and all saw-dust, chips, and other debris, thoroughly hosed out. Some specifications require that this scum be removed by the use of a stream of water under about 80 p.s.i. pressure through a $\frac{1}{8}$-in. nozzle, after the concrete has hardened sufficiently to sustain the weight of a man. The hardened concrete should be wet, but there

should be no pools of free water standing on its surface. A layer of mortar, from one to two inches thick, placed immediately ahead of the concrete, will greatly improve the bond. The excess mortar will be worked into the concrete above, and no difference in color will be apparent on the finished surface. With hand placing, especially in deep forms, the first layers must necessarily be fairly wet. As the forms are filled, and the concrete becomes more accessible and easier to spade, the mix should be stiffened to absorb the excess water as it rises. The last layer in each lift is easily accessible, and can be placed with considerably less water.

The free fall of concrete in forms for reinforced walls should be limited to three feet. Thus mortar which splashes on the forms and the steel will be covered with fresh concrete before it has a chance to dry. Otherwise, by drying on the forms, the mortar scabs will be insecurely bonded to the mass, and will either come off with the forms or, if they adhere to the concrete, will be easily removed by weathering. Mortar which dries on the steel reduces the bond between the steel and the concrete. Where there is excessive loss of mortar from the concrete, the consequent harshness of the mix frequently results in rock pockets or honeycomb. The height of free fall can be regulated by using rectangular downspouts or trunks of varying lengths, or by placing through windows in the back form.

The concrete should be placed in level layers not over 1½ to 2 ft. deep. This avoids flowing the concrete horizontally in the forms, a practice which invariably causes separation and the formation of unsightly flow lines. Care should be taken to avoid trapping pockets of thin paste or "soup," particularly at the corners. If such pockets occur, they should be drained by drilling holes in the back form. One has but to examine a few carelessly built retaining walls or exposed building foundations to appreciate the importance of these precautions. By spacing the downspouts or the windows not more than 6 ft. apart, horizontally, excessive flow can be avoided.

In summer construction, a vertical placing speed of about 2 ft. per hour will usually produce satisfactory results. For hand placing, 1½ ft. layers at 45-minute intervals are recommended. The pressure tending to squeeze water to the top increases with the depth of fresh concrete. As the concrete begins to "set," it becomes able to sustain this pressure and ceases to give up water. The depth of plastic or unset concrete depends not only on the placing speed but also on the time of set. The time of set is greater at lower temperatures, hence placing must be slowed up in cooler weather. Whatever the placing

speed, the concrete should be reconsolidated by tamping or vibrating after it has had opportunity to settle but before the next layer is placed. Water rising in the center of the wall moves slowly, because of the many obstructions. Against the forms, however, it has a more direct path, and may rise with sufficient velocity to wash cement from the sand grains in irregular channels, thus producing "sand streaks." Second to a deficiency of fine particles in the sand, too rapid placing is the most common cause of sand streaking. Sand streaks are also likely to occur in cool weather because of delayed set. While these are but surface defects, they are unsightly and can be rendered inconspicuous only by painting.

AVOID RUBBING EXPENSE—SPEND MORE ON FORMS AND PLACING

The practice of rubbing concrete surfaces to eliminate form marks and other irregularities, and to produce a more uniform color, while followed to some extent in architectural concrete construction, is not popular among architects who have made a specialty of this class of work. The general feeling is that the cost of a good rubbing job, if expended on better forms, more carefully designed mix, and greater care in placing, eliminates the necessity for rubbing. The usual procedure involves removing the forms within 24 hours, rubbing up a paste of partially hardened cement with carborundum or other abrasive blocks, and floating to distribute the paste uniformly over the surface. Because of the risk of washing off the freshly rubbed paste, water is seldom applied for curing during the remainder of the day, and frequently the concrete receives no curing at all after the rubbing operations. The cement which has been disturbed has already expended part of its ability to harden. In order to develop what is left, and to insure bonding the paste thoroughly to the concrete surface, curing is very necessary. The paste affords no protection against drying of the concrete, hence failure to cure after rubbing not only results in an imperfect surface, but seriously impairs the quality of the underlying concrete.

The rubbed surface presents another disadvantage in that, temporarily at least, it conceals defects resulting from poor form work, improper proportions, or careless placing. Where rubbing is prohibited by specification, greater care is taken to prevent the occurrence of such defects. The average rubbed surface appears less uniform after a few years of weathering than a surface of properly placed concrete which has received no treatment other than cleaning.

CURING

The laxity in enforcing curing specifications, which has characterized building construction in recent years, makes it difficult for the con-

tractor who has formed the habit of allowing little or nothing for curing in his bid on an architectural concrete job. He is entitled to know in advance exactly what will be required, and curing specifications should leave no doubt in his mind. A recent specification for architectural concrete required that after placing has been started, the full time of one man should be devoted to keeping the new concrete continuously wet for the first seven days.

Walls can be kept wet by hand sprinkling or by improvised sprinkler systems. The frequency of wetting and the interference with construction operations can be reduced by draping burlap over the wall in such manner that the surface is 100 per cent covered. Colorless coatings, whose purpose is to cure by sealing in the mixing water, are still definitely in the experimental stage.

Curing of slabs and other horizontal surfaces presents less difficulty than curing of walls. Ponding is the best curing method known, if it can be done without interfering with other crafts. A covering of burlap or sand, sprinkled frequently to prevent drying, produces excellent results. Since the mixing water itself is sufficient for curing if completely retained, a covering of waterproof paper, carefully lapped and weighted down, will serve the purpose.

Whatever method is employed, it is essential to protect concrete from drying right from the start, and to cure continuously for at least five days. Approximately 50 per cent improvement in quality can be obtained by thorough curing, as compared with no curing at all.

ARCHITECTURAL CONCRETE REQUIRES NEW SKILL AND TECHNIQUE

Best results will be obtained if owner, architect, and contractor alike will appreciate the difference in construction methods and quality of product required for architectural concrete as compared with completely protected and concealed structural concrete. Architectural concrete is comparatively new in many parts of the country, and like any new method or product, it calls for new skill and technique. We have learned the importance of durable concrete and how to produce it from our experience with engineering construction in regions of freezing winters. There is available to us the benefit of years of experience on the West Coast in developing the necessary construction technique. The sensible application of this combination of experience to architectural concrete construction will produce structures of pleasing appearance and lasting quality.

Discussion, to close in June 1939 JOURNAL, *should reach A. C. I. Secretary by April 1.*

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Low Head Permeability Tests of Mortar Pots*

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MEMBER AMERICAN CONCRETE INSTITUTE
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SYNOPSIS

The paper, "A Simple Test for the Water Permeability of Concrete," gave rise to questions of technique and interpretation of results—among them the degree of dependability on permeability values established by two tests on small, thin-walled mortar cups. In the course of an investigation of protective coatings for concrete in our laboratory an opportunity presented itself to determine the permeability of 22 mortar "flower pots," each of about 3 liters capacity, made of 1:4 mortar in the same mold on different days, and cured 7 days in the damp room. Permeabilities were determined at intervals up to 40 days, at which time the rates of water loss had become nearly constant for the individual pots and varied from 4.5 to 9 ml. per day. The conclusion is drawn that there is considerable variation in pore structure of test pots of this type, even when all reasonable care is exerted in their fabrication, and that in general, more than two specimens should be tested for establishing permeability coefficients.

When the Wiley and Coulson paper, "A Simple Test for the Water Permeability of Concrete," was published**, we were engaged in making up a considerable number of concrete and mortar pots of the type indicated by Fig. 1. These pots are of conventional flower-pot design, cast in a machined iron mold, and of an interior contour well adapted for applying paints and protective treatments, the durability of which, when in contact with any type of liquid that might be stored in concrete, could be readily investigated. The top rim of these pots is ground on a flat plate to give a bearing about 5/16 in. wide, and this permits an air tight seal to be made with a layer of cup grease and a 10 in. square glass plate. The plan was to fill these pots, after treat-

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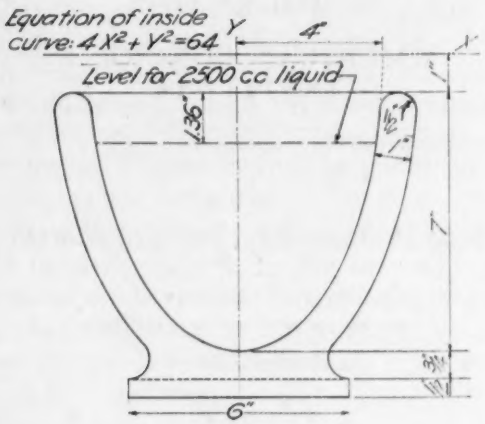

FIG. 1

ment, with the liquids under investigation, set them on the shelf, and from time to time weigh them on a Troemner scale that gives the weights to about 1 gram.[c] In this manner the failure of a coating or treatment could easily be detected by loss in weight.

After experimenting for some time with different mortars and concrete mixtures, the conclusion was reached that fairly permeable pots, with reasonably smooth interiors free from air bubbles, were the most satisfactory for tests of coatings, and eventually a technique was developed for turning out such pots from a 1:4 sand mortar with a net water-cement ratio of about 7.3 gal. per sack. The sand was pregraded to uniform sieve analysis, and the pots were given a rub with neat cement paste immediately after removal of the mold. This was not to form a coating but to fill the small cavities and give a surface to which a paint coating could be applied with fair assurance of getting a continuous film.

There were 22 of these untreated pots available when the Wiley and Coulson paper suggested the question of determining their uniformity in regard to water permeability. While uniformity in this respect was not essential to our purpose, it would nevertheless be desirable if the degree of permeability was to be determined from only two or three companion pots. These pots were made with considerable care as to uniformity of ingredients, consistency and curing (7 days in damp room) and would apparently serve to indicate the range in seepage rate that might be expected from specimens so made.

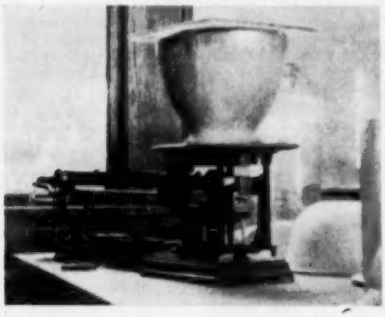

FIG. 2—WEIGHING POTS UNDER TEST AT FREQUENT INTERVALS SHOWS PERMEABILITY

Accordingly the pots were prepared by coating the rims with two coats of Bakelite varnish to prevent absorption of the seal grease, and were then filled with 2500 cc. of tap water to the level shown in Fig. 1. They were then weighed, (Fig. 2), placed on shelves in one corner of the laboratory, and reweighed daily, or sometimes at 2-day intervals, for forty days, or longer. It was realized that the dry condition of the pots when filled, and the comparatively short curing, would tend to give high permeability rates at first, but it was expected that after about 20 days the rates would be fairly steady.

The cumulative loss was plotted as shown for a few of the pots (No. 101-106) in Fig. 3. The lack of smoothness in the curves is almost unquestionably due to changes in the temperature and humidity of the air in the laboratory, particularly over week ends when the tem-

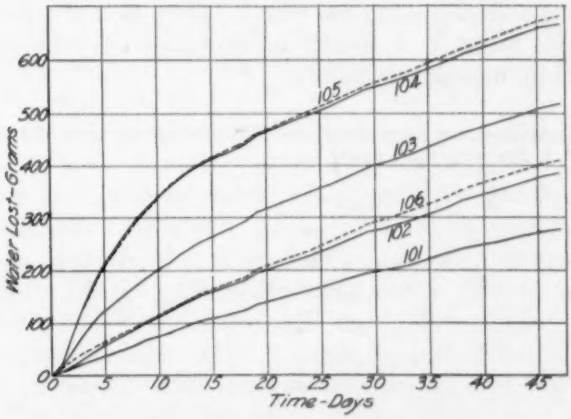

FIG. 3—PERMEABILITY OF 1:4 MORTAR POTS

TABLE 1—RESULTS OF PERMEABILITY TESTS OF MORTAR POTS

Batch for each pot: 1375 g. cement; 1100 g. each of five machine-sieved fractions of sand, 4-100; 950 cc. water. Net water-cement ratio: 7.3 gal./sack; Consistency: semi-plastic.
Rates of loss in grams of water per day at 10, 20 and 40 days after beginning of test.

Pot No.	After 10 Days	After 20 Days	After 40 Dzys	Deviation at 40 Days	Per Cent Deviation
83	19.5	11.6	7.0	+0.2	+2.9
84	13.9	9.3	6.3	−0.5	−7.4
85	14.1	9.0	6.5	−0.3	−4.4
88	10.5	8.1	6.0	−0.8	−11.8
89	11.8	7.8	6.0	−0.8	−11.8
90	20.0	11.2	7.3	+0.5	+7.4
91	16.8	10.0	7.0	+0.2	+2.9
92	20.1	10.9	7.1	+0.3	+4.4
93	16.2	8.4	5.1	−1.7	−25.0
94	19.7	10.9	6.8	0.0	0.0
95	17.8	10.0	4.4	−2.4	−35.3
96	19.2	11.3	7.9	+1.1	+16.2
97	13.6	8.6	6.5	−0.3	−4.4
98	19.7	12.7	9.1	+2.3	+33.8
99	17.5	11.4	8.4	+1.6	+23.6
100	20.6	12.4	9.1	+2.3	+33.8
101	7.4	5.9	4.7	−2.1	−30.9
102	10.1	7.6	6.3	−0.5	−7.4
103	14.2	9.4	7.0	+0.2	+2.9
104	18.9	9.7	6.9	+0.1	+1.5
105	20.6	9.9	7.2	+0.4	+5.9
106	10.4	8.8	6.9	+0.1	+1.5
Avg.	16.0	9.8	6.8	0.85	12.5

perature was lower than on week days, but these irregularities do not seriously interfere with determining the slopes of the curves at any given time.

Table 1 gives the permeability rates as taken from the curves for the individual pots at 10, 20 and 40 days. It is to be noted that the early rates of permeability are considerably higher than the rates at 40 days, which at that time seem to have become nearly constant, and to be more or less indicative of the pore structure. Nevertheless, the average deviation in permeability for the 22 pots is considerable, and indicates that with all reasonable care in fabrication of test specimens, a considerable number of similar or companion specimens must be tested to obtain dependable values.

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Job Problems and Practice

Some will ask questions — Some will answer them — Some will do both

A. C. I. members are invited to use this new JPP department as an informal means toward mutual help. When a problem holds possibilities of general interest the discussion will be briefed in these pages.

If you have a problem—present it; a question, ask it. If you know the answer to a question asked, or if you can contribute something which may help in the solution, or have reason to disagree with the answers or suggestions published, your contribution will be welcome.

If you know of an interesting problem whose solution has already smoothed the way of someone in the field, tell us about it—some other A. C. I. member may need just that information.

The "answers" are the answers of individuals to whom the questions are referred and not of the A. C. I. as an organization.—EDITOR

Roughing floor slab to bond finish? (No. 35-24)

Q—What is the best way of roughing a concrete floor slab to obtain a good bond with a heavy duty concrete topping finish? The usual practice of raking the top of the base slab leaves a good deal of poorly bonded aggregate at the surface, much of which is not loose enough to be brushed off, nor enough an integral part of the base slab to which the topping is bonded.

A—The use of a grooved roller is recommended. The roller should be about 7½ in. in diameter and 11 in. long, weighing about 75 lb. The grooves on the face should be about five-eighths inches wide and three-eighths inches deep between bands of three-eighths inch width. The concrete should be rolled just when it has started to stiffen slightly so that the three-eighths inch furrows on one-inch centers produced by the roller will not close. These furrows are cleaned out with a high velocity water jet just prior to placing the topping finish and provide an excellent mechanical bond to sound compacted concrete of the base slab.

What limits for sodium sulphate tests? (No. 35-15, Jan. '39 p. 204)

A—In the accidental dropping of a line from Stanton Walker's summarization, the fact was omitted that federal specifications fix the limit on loss in 5 cycles of sodium sulphate test at 15 per cent for coarse aggregates.

The historically minded will be interested in this contribution to the discussion:

It is frequently inferred that there is a lack of information showing the relation between the results of laboratory accelerated freezing tests of stone and aggregate by the sodium sulphate or Brard tests and actual results in structures. This is generally true, but it is possible to cite a notable structure, the erection of which started in 1848, and in which the stone used was subjected to this test. This is the historical brown stone Smithsonian Institution in Washington.

According to an old publication of the Maryland Geological Survey (the leaf containing the date of publication is missing unfortunately), Dr. Charles G. Page and Walter R. Johnson made a number of tests, including the Brard tests, of certain sandstone occurring along the Potomac River west of Washington. Near where the Seneca Creek flows into the Potomac there existed a deposit of light to dark red sandstone, known as Seneca sandstone. This had been quarried as early as 1774 and used in a canal around Great Falls in the Potomac River. Later it was used in the aqueducts, locks, and other structures in the Chesapeake and Ohio Canal.

With such a history it was natural that it should be considered in the selection of stone by those having charge of the erection of the first building of the Smithsonian Institution. Accordingly, the Page and Johnson tests were carried out and reported in the American Journal of Science, Vol XI, 1851, page 7, and in a Report of the Board of Regents of the Smithsonian Institution: Senate Document; 30th Congress, First Session; No. 23, pages 21-22.

The original papers have not been consulted, but in the old Maryland Geological Survey Report, the following is stated:

"The weight and disintegrating effects of 'frost' upon the Seneca sandstone were carefully studied by the Brard method before the stone was accepted for the Smithsonian Institution, and we have as a result of Dr. Page's investigation the following determinations:

	Lost by frost in Grains
Dark red Seneca sandstone (similar to Peter's)	0.70
Light Seneca sandstone, dove colored	1.78
Dark coarse sandstone, of Seneca Aqueduct, Peter's Quarry	5.60
Sandstone four miles above No. 2 D, Peter's next west Beaver Dam Quarry	1.58
Dark sandstone, from quarry near Woods' residence	3.94"

The above figures refere to the loss in the Brard test rather than by a water freezing test. Note that the loss is expressed in grains without any reference to the number of cycles referred to nor is the original size of the specimens given.

It also may be of interest to add that if one may judge aright from the text, the stone from the Peter's Quarry was that used in the construction, in spite of the fact that it gave the highest loss in the test. However, the present condition of the Smithsonian Institution, the Cabin John Bridge on Conduit Road, Washington,

and the masonry locks, etc., on the Chesapeake and Ohio Canal, indicates a very serviceable stone, no matter what the results of these sodium sulphate tests may have been.—P. H. BATES.

Field curing room (No. 35-25)

The curing room at Boulder Dam was equipped with refrigeration and air conditioning equipment costing several thousand dollars. It provided for keeping the humidity at about 90 per cent and the temperature at 70° F. At Coulee Dam a novel arrangement for temperature and humidity control was tried which is very inexpensive.

Small grocery type vegetable sprays were installed above the curing racks, and until the hot part of summer, sufficient water, cooler than 70° F., was atomized into the room to keep the temperature down to 70° F. In the summer when the water finally warmed up to 68° or more, and the air outside was as high as 108°, the curing room temperature gradually rose to above 70°. An opening was then cut in the outer wall and a fan was placed to blow hot, dry, outside air into the fog room. This immediately cooled the room down to the required temperature and by manually opening or closing the outside air opening the temperatures have been very easily controlled, to within 1° F.

In cool weather electric strips or glow heaters are turned on or off manually to keep the desired temperature.

The room holds 1000, 6 by 12-in. cylinders and has 8-in. rock wool insulated walls and ceiling.—O. G. PATCH.

Should reinforcement be free from rust? (No. 35-26)

Q—Should cold drawn wire mesh reinforcement be kept free from rust when it is to be used in concrete pavement slab?

A—No. The wire should be allowed to rust, at least slightly, to provide bond. Cold drawn wire has a very uniform, smooth surface and we have succeeded in slipping two feet of such wire (new) through good quality concrete, shearing two cross wire spurs, whereas the same wire bonded well when permitted to rust even superficially.

—F. V. REAGEL.

Concrete tanks for wine and grapejuice? (No. 35-27)

Q—Is it necessary to line concrete tanks for the storage of grape juice or wine in order to prevent the concrete spoiling the color and taste of the beverage?

A—Concrete tanks have long been used for the storage of grape juice and wine. The early ones were lined with glass, cemented in

place with sodium silicate. This type of liner is very expensive, not only in the original installation, but the replacements, due to breakage of the glass. A number of years ago at the request of some of the producers of grape juice, the Research Laboratory of the Portland Cement Association carried out a few experiments to determine a suitable treatment for the surface of the concrete. The two most successful treatments were emulsified asphalt and several applications of tartaric acid applied to the concrete surface. The tartaric acid reacts with the concrete, forming a coating of calcium tartrate. After a certain amount of this coating was formed, no further action appeared to take place, and the calcium tartrate did not affect the color or the taste of the grape juice stored in experimental tanks.

A questionnaire was sent to a number of companies where tanks were known to be in use for storage of wine or grape juice to learn what types of treatment were used and the success of the tanks. One company, which has 14 tanks in use, with capacities up to 25,000 gal., reported not a single leak, tanks giving perfect satisfaction. These tanks had been given three different treatments: paraffin, emulsified asphalt, and a mixture of alumina cement and silica sand. These tanks have been in service from 1 to 4 years.

Another large user of concrete tanks has not had an entirely satisfactory experience. The tanks had been put into service before they were sufficiently hardened and they were of unusually large capacity, ranging from 80,000 to 110,000 imperial gallons. Based on this experience they recommended the use of batteries of smaller tanks.

—A. G. TIMMS.

What heat will concrete withstand without deterioration? (No. 35-28)

Q—Will 1:2:4 concrete in precast slabs withstand without deterioration a temperature of 300° to 500° F. over heat treating furnaces? Would Haydite concrete have any advantage?

A—No coordinated experimental data are available from which it is possible to predict the expected life of concrete under all temperature conditions. Predictions must therefore be based largely on experience, supplemented by the results of various investigations.

It may be said that at temperatures of 300° to 500° F. concrete that is of good quality, that is, has been properly made and adequately cured before exposure, has about the same life expectancy as concrete exposed to normal temperature.

Dehydration of cement in concrete ordinarily is so slow that the effect is not noticeable unless it has been exposed to temperatures of

600° F. or higher over an extended period. Even at this temperature, the products of dehydration at the surface form good insulation and delay the rate of further dehydration in the interior. The life of a structure exposed to these temperatures would therefore depend largely on the mass of concrete and area of surface in exposure as compared to the mass. It is generally assumed that constant exposure to temperatures of 900 to 1,000° F. would in time cause complete dehydration and it would therefore be necessary to protect concrete with insulation where such high temperatures are encountered.

So far as life expectancy is concerned, it is doubtful if Haydite has any appreciable effect on thin sections of concrete at temperatures under 500° F. On thick sections the lower heat transmission of Haydite would greatly prolong the life of concrete even when exposed to the very high temperatures.

Precast units offer the opportunity to produce concrete of the very highest grade. They can be cured under the best conditions before exposure. Such units used in roof construction should serve their purpose indefinitely without deterioration due to exposure to temperatures under 500° F.

Metallization of concrete tanks? (No. 35-29)

Q—Can you supply references on the metallization of concrete tanks for the storage of sulphuric acid.

A—The process referred to evidently is the metal-spraying process which has been commercialized by a number of firms both in America and in Europe. It originated in Europe and is usually referred to as the Schoop process. Most of the published information on the subject has come from those interested in promoting the process. Probably the best English book on the subject from an impartial source is "Metal Spraying," T. H. Turner and N. F. Budgen (Charles Griffin & Co., London, England. The latest book (1935) is in German: "Handbuch der Metallspritz—Technik," Dr. Ing. M. U. Schoop and C. H. Daeschle (1935, Rascher & Cie, A-G, Verlag, Zurich, Leipzig, Stuttgart and Wien).

Stone sand as concrete aggregate? (No. 35-30)

Q—From the engineer in charge of a large construction project in the south came an inquiry which opens up the subject of the use of crushed stone as fine aggregate—"stone sand" in place of natural sand.

A—The answer to the inquiry is not simple. Previous to very recent years stone sand (then ordinarily known as stone screenings,

containing larger percentages of dust than the improved stone sand) was used only occasionally as a fine aggregate. Due no doubt to the fact that a complete solution had not been found in the early years for the use of crushed stone as fine aggregate, some of the early service records have not been satisfactory. Studies of some of such unsatisfactory work have shown in some instances that the screenings were made from an unsound rock; or that the particles were not the right shape or were not properly graded; or that the concrete proportions were not suited to the type of material employed.

In 1936 the National Crushed Stone Association summarized the manufacture and use of stone sand in its Bulletin No. 10 in which it was explained that the harshness found with some stone sands is due to the shape of the particles. In general a cubical shape has given satisfactory workability, whereas flat or elongated grains were likely to give harsh mixes requiring either more water per sack of cement or increased cement factor for a workable mix. The Bulletin brings out the point that cubical grains are being produced with a variety of equipment, such as cone crushers, ring rolls, rod mills, hammer mills, etc., and that the type of equipment which is suitable for any particular setup is dependent upon the characteristics of the rock to be crushed. Thus the equipment found to be satisfactory in one locality with a definite kind of stone is not necessarily satisfactory where the stone has different characteristics.

With the recognition that the problem of producing fine aggregate from crushed stone is not a problem having a single answer, procedure to solve the problem is definitely better, so that better results with stone-sand may be looked for as these improved methods and the knowledge of the variability of the problems are better known.

In the construction of Norris Dam near Knoxville, Tennessee, dolomitic limestone was used to produce stone sand for fine aggregate with apparently satisfactory results.

Through the courtesy of the Tennessee Valley Authority, Knoxville, Tenn., we quote portions of a summary of the steps in the procedure to produce the fine aggregate for Norris Dam:

Prior to the selection of the crushing machinery for the production of fine aggregates for Norris Dam, samples of quarry rock were sent to several crusher manufacturers for testing. These rock samples were reduced to pass a No. 4 mesh screen in various types of crushers, including roll crushers, rod mills, modified gyratory crushers and hammermills. The resulting products were screened and the gradation curves plotted, and samples of the classified sand for each machine were compared as to the shape of the individual particles.

All of the gradation curves followed the same general trend. There was a surplus of the coarse sizes (plus 8 and plus 14 mesh) and a deficiency of the finer sizes (plus

28, plus 48, and plus 100 mesh) when compared to the desired gradation of nearly equal amounts of each size. From this viewpoint, therefore, there was little or no choice as to which machine should be used. It was clearly indicated that improvement in gradation could be effected only by selective screening and recirculation of undesirable oversizes.

A marked difference was apparent in the grain shapes of the products of the various machines. The hammermill product was cubical, the products of the gyratory crushers were wedge shape, pyramidal ,and flat, while the product of the rolls was composed of splinters. As a cubical shape is the most desirable, the hammermill product was superior. The grain shape of the material from one of the gyratory crushers was good; although not cubical, the pyramidal grains were not elongated to a marked degree.

It was claimed that the particle shapes from the cone crusher would be greatly improved under an operating condition where the discharge opening of the crusher was larger than the maximum size of grain in the product desired and the oversize screened out and recirculated for recrushing. The crusher must be choke-fed; that is, the annular feed opening must be filled to capacity. A considerable amount of the reduction would result from attrition grinding or crushing between particles as against crushing between the metal surfaces of the concaves, the latter crushing condition producing flats and undesirable shapes. This claim seemed supported in a plant where a cone crusher was being operated under these conditions. Consequently, the cone crusher was strongly considered for sand production, due largely to its freedom from repairs when compared to the attention required by a hammermill.

To investigate further the use of this type equipment, a three-foot Symons cone short head crusher was rented and installed at the quarry. Tests on it were carried out with specific information being obtained on:

1. Gradation of the sand, or that portion of it passing a No. 4 mesh screen, with special attention to the part passing a No. 100 mesh screen, and also with the change in gradation, if any, due to changes in feed condition or load on the crusher.

2. Quality of the sand; that is, the shape of the individual particles or grains.

3. Amount of the circulating load, or that oversized portion of the crusher's product returned for further grinding.

4. Capacity of the crusher as a whole, and the capacity to produce sand passing a No. 4 mesh screen.

Description of plant

In general, the plant layout consisted of the crusher, two belt conveyors, and a screen. These units were so arranged that the crusher could be operated in a closed circuit. The feed for the crusher was taken from a bin and consisted mostly of 1½- and ¾-in. stone. From the bottom of the storage bin the first conveyor transported crusher feed to the double deck screen located over the crusher. Material from the crusher was carried by the second conveyor and deposited on the first conveyor. In this manner new feed and crusher products were elevated together to the screen where by various chute arrangements the sized materials could be sent through the crusher or to the sand storage bin.

All oversize from the upper deck of the screen was dropped into the crusher while material passing the lower deck was chuted to the sand bin. That material retained on the lower deck was sent either to the crusher for further reduction or to storage as finished product. An adjustable blade gate at this point permitted separation in any desired proportion.

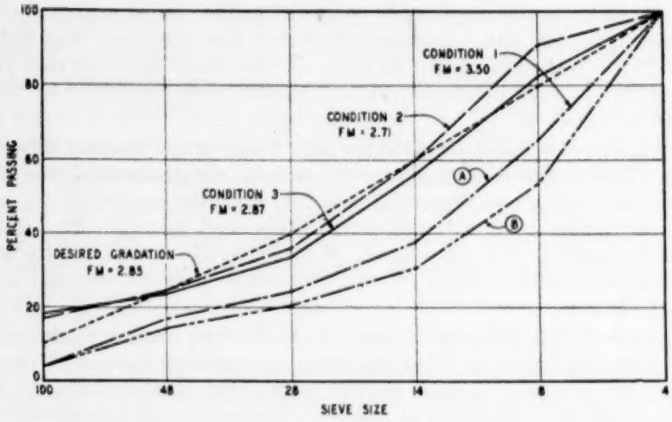

FIG. 1—SIEVE ANALYSIS OF SAND PRODUCED WITH A THREE-FOOT SYMONS CONE CRUSHER

Gradation and particle shape

The gradation of the product varied with the load on the crusher, the best gradation being obtained when the crusher was choke fed. Curve A, (Fig. 1) shows gradation for products under a choke feed and Curve B shows product with light load conditions (less than half capacity) for the same setting. Further investigation showed that the shape of grain varied with the load and the setting of the crusher. The best shaped particle was obtained when the crusher was set at about $\frac{3}{8}$-in. opening and choke fed. Smaller settings produced flat particles. The grain shape was not nearly as cubical as the one produced by hammermills. It had a greater proportion of wedge-shape particles than cubes. A comparison of the two products shows the preponderance of slab particles in the cone crusher product.

The plant was operated at full capacity under three conditions, differing only as to arrangement of screen cloths and the size of mesh used. A screen analysis curve of the products with each condition is shown as compared to a desired gradation curve in Fig. 1.

1. The screen was operated as a single screen with a cloth having a 0.194-in. opening.

2. In an effort to reduce the proportion of the larger sizes a screen cloth having a smaller opening (0.145 in.) was tried.

3. The first two tests indicated that a better sand could be obtained by the use of a double deck screen so arranged that the proportion of the coarser sizes is controllable to a degree by selective screening and recirculation. The upper deck was fitted with a cloth having a 0.194-in. opening, and the lower deck with one having an opening of 0.095 in. The resultant product was composed of all sand passing the lower deck, plus one-third of that passing the upper screen. This condition gave the best results obtainable with the plant setup.

Crusher capacity

Measurements were made of the crusher output by diverting the material on the conveyor belt into sample boxes for a noted time and converting to tons. The capacity for condition 1 above was 58 tons per hour at $\frac{3}{8}$-in. setting with the feed consisting

of one-third new feed and two-thirds circulating load. Under this condition the amount of material produced passing a No. 4 mesh screen was 20.5 tons per hour. For the best graded sand as in condition 3, the system output was 18.6 tons per hour.

Workability tests

Tests were made on hammermill and cone crusher sand to determine the amount of cement paste required to produce the same workability. The hammermill sand was the product of a mill in a local cement plant several truckloads of quarry stone having been hauled to the mill for crushing. Cone crusher sand was obtained from the experimental crushing plant.

Two methods were employed to determine the relative workability of the mortars made with the two sands. One method was the use of the flow table, and the other was the use of an improvised rig, called the "pendulumeter." This latter instrument consisted of a sharp blade fastened to a steel ball suspended so as to swing in a constant arc. The mortar to be tested was deposited in a trough, leveled, and placed in the path of the pendulumeter. The ball was pulled back to a fixed point and released. The point of the blade raked through the mortar for a certain distance, depending on the plasticity of the mortar.

Tests were made with mortar of different mixes but with constant water-cement ratio. Therefore, for two mixes of the same proportion of cement paste to aggregate, any difference in plasticity or workability would be due to the nature of the sand in the mortar. Each sand had been screened and recombined to the same gradation.

The following table shows the result from the flow table and the pendulumeter tests for mortars mixed in varying proportions with a cement paste of a constant water-cement ratio (0.6 by weight).

Mix No.	Flow Table Tests		Pendulumeter Tests	
	Cone Crusher	Hammermill	Cone Crusher	Hammermill
1	96.3%	108.3%	34.3	34.8
2	78.3%	84.1%	26.3	27.3
3	53.0%	59.8%	19.0	23.5
4	29.6%	48.9%	13.5	16.6

Mixes were then made using more cement paste with the cone crusher mortar than with the hammermill sand in an effort to get the same percentage flow. These indicated that the same percentage flow could be obtained with the addition of about four to five per cent more cement paste to the cone crusher sand mortar.

Further tests were made on the two sands using a concrete instead of mortar. The sands were screened and recombined to the same gradation as before, and mixed with equal amounts of ¾-in. gravel of the same gradation. The batches were mixed with varying amounts of cement, using constant water-cement ratio. These tests substantiated the results from all other tests in that concrete made with sand produced by a cone crusher requires more cement for the same workability than a concrete made with sand produced by a hammermill.

Comparison of plant costs

Capacity tests of the 3-foot cone crushers indicated that two 5½-foot machines would produce sand at the desired rate. These two machines with motors and accessories would cost about $36,000. Two hammermills with equivalent capacity with motors and controls would cost about $12,000. The plant layout and other machinery units necessary to complete the installation for sand manufacture would be the same, whether cone crushers or hammermills were used for the reduction units. The

$24,000 difference in cost between the two installations when applied to the total sand requirement of 570,000 tons provided 4.2 cents per ton for hammer maintenance and repairs.

Summary of investigation

Reviewing the points influencing the decision to use hammermills instead of cone crushers to produce sand:

1. A cement saving of from four to five per cent was indicated if hammermill sand were used instead of cone crusher sand.

2. The lower first cost of the hammermills would, in a large part if not wholly, pay for hammer renewals.

3. The gradations of the products of the two mills were substantially the same, indicating that for a better graded product selective screening was necessary. The material passing a 100-mesh sieve was expected to be somewhat more with the hammermill product, resulting in additional waste through washing.

The foregoing is presented rather less for its value as a typical solution of the problem of making stone sand than as an example of the nature of the problem and the difficulty of a simple solution.

Concrete tanks for oil and gasoline? (No. 35-31)

Q—What is the suitability of waterproofed concrete for petrol tanks. We have been unable to locate in the JOURNAL, since the writer became a member, any reference to this subject. The tanks constructed would be placed below ground. We have had a great deal of experience in waterproof tank construction, also tanks to contain oils and fats in which we experienced no trouble.

Where resistance to water only is required we apply an integrally waterproofed cement render. In the case of oils and fats we use a similar method, finally treating the surface with three coats of solution of magnesium and zinc fluo-silicate. With petrol, however, volatility is so high that special precautions would probably be necessary. We think that the application of an integrally waterproofed cement render applied in two coats, with the second coat containing a proportion of ground iron in lieu of an equal quantity of aggregate, followed by a penetrative coating of petrol-insoluble material, would be suitable. We are constructing a test tank on this basis. Any information or references would be appreciated. (Inquiry from an A. C. I. member in Australia).

A—Reference is made to "Tests of Concrete Tanks for Oil Storage," by Pearson and Smith, *Proceedings* Amer. Concrete Inst., Vol. 15, 1919, p. 186, and "Further Tests of Concrete Tanks for Oil Storage," by Smith, Vol. 18, 1922, p. 22. As a result of the tests described in these papers—which were made at the Bureau of Standards—it was concluded that the leakage of the concrete that was least permeable

to oils would probably be sufficient that the tank would prove uneconomical unless some impervious coating could be found. The lightest oils that can be stored are apparently those with a specific gravity above .875 or 30° Bé. Sodium silicate and similar coatings were not found effective in reducing the permeability to lighter oils. The most promising material for coating the concrete to reduce the permeation was spar varnish, two coats being given in the test. The papers state that the retaining value of concrete tanks for oils must depend fundamentally upon the concrete itself and probably no coatings short of an all-metal lining will be found capable of converting a *poor* tank into a *good* one.

The problem of making a tank impermeable to gasoline is somewhat similar to making it impermeable to water, the concrete being simply more permeable to gasoline than to water. Coatings of mortars rich in cement have been found to reduce permeability to water and act similarly with gasoline, but will be somewhat less effective. The only method of making a concrete tank completely impermeable to gasoline would be to coat it with an impermeable membrane. As previously stated, spar varnish appeared to be the most promising material, other than a metal lining for the tank.

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Current Reviews

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Earthquakes and structures

The late L. M. HOSKINS and JOHN D. GALLOWAY, *Proc. Am. Soc. C. E.*, Vol. 64, No. 10 (Dec. 1938), p. 1993-2021. Reviewed by H. J. GILKEY

Earthquake waves are described and reference is made to their cause, complexity and characteristics. The nature of the ground motion is used as a basis for advancing some fundamental rules and conclusions. Some consideration is given to the case of an elastic column sustaining a load, as an extension of the theory. The treatment is limited to no particular type of structure.

Unusual concrete rigid frame test

Engineering News Record, Vol. 121, No. 20, Nov. 17, 1938, pp. 615-617. Reviewed by ROBT. W. BEAL

It became necessary recently to demolish a well-designed, reinforced concrete, skew, rigid-frame bridge at Glendale, California, in connection with a flood control project. Several engineers took strain and deflection measurements as the structure was loaded on a three-foot wide transverse central strip. A total of 430 tons was loaded onto the bridge, which had a span of 43 ft. and a width of 60 ft. Measurements were also taken as the load was removed unsymmetrically. Deflection and strain curves are shown for many points in the structure.

Simplified wind-stress analysis of tall buildings

OTTO GOTTSCHALK. *Proc. Am. Soc. C. E.*, Vol. 64, No. 10 (Dec. 1938), p. 2023-2032. Reviewed by H. J. GILKEY

The method of analysis is based upon model experiments and upon studies leading to purely geometrical interpretations of them. A structural frame is analyzed by subjecting it to given displacements and by multiplying these by the given loads for translation into stresses. The method by-passes the need for the solution of numerous abstract equations. It applies to frames of any number of stories with any number of bays and with beams and columns having any constant section.

Concrete surfaces

Bautechnische Mitteilungen, No. 2, pp. 1-4, June-Sept. 1938. Reviewed by F. L. EHASZ

Three poor examples of concrete surfaces on piers, frames and walls, showing ugly streaks, spots and signs of efflorescence, are shown to be in sharp contrast with several good ones. The latter acquired an aesthetic appearance as a result of rapping,

scraping and pointing. Variations in the grain-size of surfaces may be effected by control of the extent of these processes. Moldings and retaining walls, in addition to frames and piers, are illustrated to show the pleasing effects obtainable.

The effect of sand on the durability of mortar

BERT MYERS (Iowa State Highway Commission) Highway Research Board paper.
HIGHWAY RESEARCH ABSTRACT

This report shows the loss of strength due to freezing and thawing of mortars made from three brands of cement each combined with five sands of different lithological composition. Results indicate that the relative durability of cement as indicated by freezing and thawing tests of mortar will depend upon the kind of sand used, and that the relative durability of sand as indicated by the same test will depend upon the cement used.

Concrete-stone industry and municipal street construction

SCHARNHORST, OPPELN, *Betonstein Zeitung*, V. 4, No. 23, pp. 347-350, Dec. 10, 1938.
Reviewed by F. L. EHASZ

The great interest felt toward street construction today and the extent to which municipalities go in their development are pointedout. More consideration toward concrete-stone products is suggested. Sidewalk slabs, curbs, borders, lamp-posts and towers are the chief items mentioned as having excellent qualities. Closer observation of DIN terms of delivery of products and a regulated apprenticeship for the development of expert workers are to be aspired to.

Iron inner tube strengthens conduit

R. L. PRICE, *Engineering News Record*, Vol. 121, No. 16, Oct. 20, 1938, pp. 495-498.
Reviewed by ROBT. W. BEAL

Construction details are given in connection with the lining with iron plate of 500 ft. of 72-in. concrete water conduit in Toledo, Ohio. The 26-year old conduit is the sole supply line to the city, and all work had to be done during limited periods during the night when the line could be dewatered. The plates were bolted into place and pressure-grouted with a rich mortar of quick-setting cement, so that placing of the mortar could continue within 30 minutes of the time when it was necessary to fill the conduit with water.

New top pressed on concrete road

GUNNAR JOHNSON, *Engineering News Record*, Vol. 121, No. 17, Oct. 27, 1938, pp. 531-532.
Reviewed by ROBT. W. BEAL

A short stretch of concrete road has been resurfaced in Pennsylvania with only a 2 in. topping. The thin surface is applied under about 1500 lb. per square ft. pressure with the use of vacuum mats. It is felt that the 2 in. thickness will be sufficient if the pressure used will cause the finish to bond with the old slab to form essentially an integral part of the roadway, rather than a separate layer. The experimental section of road contains several hills and curves. Details of preparing the old slab and placing the thin surface are given.

Ore-processed gravel for concrete

Engineering News-Record, Vol. 121, No. 22, Dec. 1, 1938, pp. 699-702. Reviewed by S. J. CHAMBERLIN

The contractors for the Marshall Ford dam used ore classifiers, scrubbers and rod mills to produce five sizes of aggregate. The gravel bar material excavated by dragline and moved to the raw material stock pile by trucks and belt conveyor is elevated

to the top of the processing and classification plant. The sand separated by screening passes through a cone hopper to the classifier which separates it into sizes by flotation. The plus 14 material is reduced in size by the rod mill. The larger aggregates are cleaned of dust and slime by a rotating scrubber screen. An aerial tramway conveys the material to bins for the respective sizes at the dam.

Design of dowels in transverse joints of concrete pavements

BENGT F. FRIBERG, *Proc. Am. Soc. C. E.*, Vol. 64, No. 9 (Nov. 1938), p. 1809-1828.
Reviewed by H. J. GILKEY

In an attempt to put the design of dowels on a rational mathematical basis, relatively simple expressions are derived on the basis of assumed values for some of the constants of the materials. It is concluded that by proper dowel design stresses can be reduced 25 per cent to 35 per cent at the average joint opening. One inch diameter dowels may be expected to deflect only about half as much as 0.75 in. dowels. One half the usual 24 in. length for dowel bars is ample. Spacing is important and can be regulated to secure any desired stress relief within the available limits. The proper alignment and placing of the dowels is extremely important.

The carrying capacity of reinforced concrete members subjected to flexure

F. GEBAUER, *Beton und Eisen*, Vol. 37, No. 21 (Nov. 5, 1938), p. 342. Reviewed by A. U. THEUER

The author presents a critical review of the German Reinforced Concrete Committee Report No. 85, submitted under the direction of Dr. Friederich. At some length, reasons for rejecting the arguments presented by Dr. Friederich favoring the retention of the modular-ratio in practice in spite of all the theoretical shortcomings of this procedure, are discussed. The design methods as developed by the author himself (presented in a series of articles in B. u. E. during 1936) are then reviewed leading to the conclusion that for any desired factor of safety, under known conditions of equilibrium for fracture, for a given concrete cube strength, and for a given steel yield point stress, a simple and direct manner of design, independent of the modular ratio, exists.

Thirteen armory buildings

S. E. BERKENBLIT, *Engineering News-Record*, Vol. 121, No. 22, Dec. 1, 1938, pp. 706-707.
Reviewed by S. J. CHAMBERLIN

New National Guard armories in Illinois are being constructed of concrete by the WPA because of the relatively larger amount of common labor, rather than skilled that can be used. The two-hinged, concrete arch bents of 90 ft. span have their legs connected by tension rods under the drill hall floor. Monolithic brackets support concrete purlins which carry the wood sheathing and roofing. Carefully planned construction joints in the bents and monolithic walls were at the one-story roof levels, the top of the window sash above, and at the inclined rafter of each bent halfway between haunch and apex. A timber traveler carried a complete form system for two adjacent arches and connecting purlins, and a concrete hoist tower.

Contribution to the question of the constitution of portland cement. III

K. KOYANAGI, S. KATCH and T. SUDOH. *Zement*, Vol. 27, No. 50, Dec. 15, 1938, pp. 785-788.
Reviewed by L. T. BROWNMILLER

The authors report here a series of experiments in which the combination of the magnesia of portland cement is investigated. Three high magnesia clinkers were

prepared containing 4.33, 5.70 and 11.44 per cent MgO. The clinkers were finely ground and separated into fractions of varying specific gravity by centrifuging. Chemical analysis and x-ray diffraction patterns were made of the various fractions. The lines of MgO were present in diffraction patterns of all the fractions; the intensity of the lines varied with the mount of MgO present. From their experiments they conclude that the MgO is principally present in the free state in normal portland cement clinker as well as in high MgO clinker. Their data also indicate that the fractions of high specific gravity contain the higher amounts of MgO.

The field for low-heat cement

H. S. MEISSNER and W. T. MORAN, *Engineering News-Record*, Vol. 121, No. 19, Nov. 10, 1938, pp. 589-593. Reviewed by S. J. CHAMBERLIN

The article presents a brief review of the factors leading up to the introduction of low-heat cement, the researches on which it is based, and gives the experiences gained in its employment and production. Extensive research by the Bureau of Reclamation on the factors influencing volume changes and consequent cracking of mass concrete resulted in the production of low-heat cement. Improved chemical composition, by decreasing the tricalcium aluminate and tricalcium silicate, with increased fineness resulted in a cement with decreased heat of hydration, satisfactory strength, and increased workability. The rate of heat liberation is lower and extends over a longer period. Some changes in design and construction are necessary in the use of low-heat cement, particularly the curing period should be prolonged because the gain in strength is slower. Use of low-heat cement in Morris, Boulder and other Bureau dams has shown decreased cracking. The cement has been produced in eight large cement plants, without material changes in equipment or methods of operation, at a cost slightly more than for standard.

Influence of the method and length of mixing on the characteristics of concrete

LeGenie Civil, Vol. CXIII, No. 19, Nov. 5, 1938, pp. 392-393. Reviewed by R. L. BERTIN

The monograph gives the compressive resistance, flexural resistance and permeability at various ages of concrete specimens of various proportions and cements (high alumina and ordinary portland) mixed by the following methods:

(a) Five minutes in a mechanical laboratory mixer.

(b) Mixed by hand with

(1) a trowel for periods of 3, 5, 10 and 15 minutes.

(2) shovels, 5 and 10 minutes.

(3) rakes, 5 and 10 minutes.

The following conclusions are drawn from the test results:

(1) High alumina cement produces concrete which is less permeable than that made with ordinary portland cement.

(2) For either cement, concrete mixed by hand shows inferior strength and water tightness at early ages. These inferiorities practically disappear at the age of 90 days, provided the hand mixed concrete has been mixed sufficiently.

The effect of water in concrete

GABRIEL A. ASHKENAZI, *Cement and Lime Manufacture*, Vol. XI, No. 11, Nov., 1938, pp. 237-241. Reviewed by J. C. PEARSON

Author reviews briefly the prevailing notions of the mechanism of setting and hardening of cement, pointing out that the water needed for decomposition of cement and for production of gel is probably less than half that normally used in mixing the

concrete, even after long curing or aging. Part of the excess water is lost by evaporation, part remains absorbed in the colloidal cells, and parts as free water in cavities and pores. A very wet mix produces an over-expanded gel diluted with water. Under unfavorable conditions of humidity and temperature, desiccation may proceed rapidly, with sudden shrinkage. This may result in cracking of the gel and the expulsion of water, and the power to reabsorb water later may be reduced or lost. This leads to a conception of the internal structure, not as a stone-like solid, but as a network of interstices separating the really solid ingredients which form its skeleton. Naturally the permeability of such a structure is greater, the greater the amount of excess water. The effect of carbonation, which the author pictures as a sealing and protective action in a dense, relatively dry mix, may be entirely lost in a structure which is sufficiently open to permit ready movement of free water. The conclusion is that the use of wet mixes cannot be justified.

Load and deflection characteristics of dowels in transverse joints of concrete pavements

BENGT FRIBERG (The Laclede Steel Company) Highway Research Board paper.
HIGHWAY RESEARCH ABSTRACTS

Laboratory tests on dowels encased in concrete indicate 1-in. dowels and possibly ¾-in. dowels to constitute effective agents of load transfer across transverse joints in concrete pavements of common thicknesses. The ¾-in. dowels probably should be limited to pavements subjected to single wheel loads of not over 4,000 lb. including impact effects.

Initial failure at dowels occurs by spalling of the concrete at the face of the joint; 50 per cent higher load may be taken by the dowels before ultimate failure. Beyond the initial failure the dowels are ineffective for stress relief, but may still serve a useful purpose of pavement alignment.

Both initial and ultimate strength of ¾-in. dowels and the ultimate strength of 1-in. dowels increase with increasing yield point of the dowel steel.

The modulus of dowel reaction increases with increasing concrete strength and decreases with increasing concrete depth below the dowels. For 6-in. and 7-in. pavement with the dowels subjected to vertical loads, the value of K most applicable to pavement designs is 1,000,000 pounds per cu. in.

New test for workability

Commonwealth Engineer, Oct. 1938 (Australia) HIGHWAY RESEARCH ABSTRACT

A recent editorial in *Highway Research Abstracts* on Cement-Concrete Mixtures (R. W. Crum, May, 1938) pointed out that one of the stumbling blocks in the way of a real design of concrete mixtures is the lack of a test for workability. A new test is proposed by Jean Faury, a French engineer, proposed to imitate the job conditions of tamping and vibrating. The test is divided into two sections, and two sets of apparatus are used. The first apparatus consists of a box with an opening in one side against which a grillage of bars abuts. The box is filled with concrete which is then tamped or vibrated a definite amount. The concrete flows through the opening and covers a semi-ellipse on the base board. The semi-axis minor of the ellipse is a measure of the plasticity of the concrete.

The apparatus used in the second test consists of two similar molds, one empty and one containing a cage of reinforcement. The latter mold is never completely filled, since there are always air pockets caused by the obstruction due to the reinforcement. The action of the walls of the box also tends to produce this result. From the known volumes of concrete filled into the two molds and the weight of the same concrete

required completely to fill the molds after the test has been made, it is possible to calculate the apparent specific gravity. The plasticity is the ratio of the apparent specific gravity of the concrete in the mold without reinforcement to that of the concrete in the second mold.

Concrete as a protective material against high-voltage x-rays

GEORGE SINGER, LAURISTON S. TAYLOR and ARVID L. CHARLTON, *Research Paper RP1155*—National Bureau of Standards, Vol. 21, Dec. 1938. AUTHOR'S ABSTRACT

A description is given of relative X-ray-transmission measurements on a group of especially prepared concrete specimens, and commercial building blocks selected to sample the concrete mixes and cover the range of concrete densities in common use. It was found that the lead equivalent of any concrete was an increasing function of its mass per unit area and independent of the nature of the mix. Relations between lead equivalence, density, mass, and thickness are given, from which the thickness of concrete necessary for adequate protection can be calculated for any voltage in the voltage interval 200 to 400 kv.

Since the protection coefficient of concrete increases rapidly with increasing excitation potential, the thickness of the concrete barrier which will provide adequate protection at say 400 kv., is not very much greater than that required to give the same degree of protection at a much lower voltage. It was found that for the building blocks tested, a barrier about 30 cm. (11.8 in.) is adequate at 400 kv., while at 200 kv. the thickness required is about 22 cm. (8.7 in.). Similarly, the thickness of concrete required at 400 kv. is about 26.5 cm., while the required thickness at 200 is 22 cm.

In both cases the additional thickness required in going from 200 to 400 kv. is small and a barrier providing adequate protection at a given excitation potential will also be adequate at a lower voltage.

The distribution of strain in the concrete of pullout specimens

H. J. GILKEY, S. J. CHAMBERLIN and R. W. BEAL (Iowa State College), Highway Research Board paper. HIGHWAY RESEARCH ABSTRACT

In the 1937 tests of beams and pullout specimens for bond resistances, extensive measurements of concrete surface strains were made with Martens mirror extensometers arranged in slightly overlapping 4-in. gage lengths along both the tensile and compressive faces of beams and along opposite faces of pullout specimens. Crack formation and growth were noted. For the beams and beam-type specimens deflections were also recorded for all increments of load. It was possible to include only a hint of these data in the report presented a year ago which appears as Chapter VIII of the High Elastic Limit Steel Studies and is published in Volume 17 of the Proceedings. This year's paper presents the evidence secured on strains, cracks and deflections for the two strengths of concrete (3,000 and 5,000 p.s.i.); three diameters of plain and deformed rail-steel bars (⅜, ⅝, and ¾-in. round); and for several steel percentages and loadings. In pullout specimens, lengths of embedment varied from 3 to 24 ins. In several respects the plotted data are strikingly consistent. Because strains were measured only at concrete surfaces it is impossible to say just what was happening at the junctions of steel and concrete. The tests, therefore, leave some important questions unanswered relative to the distribution of bond stresses along an embedded bar. The evidence does demonstrate and emphasize the progressive nature of bond resistance and also the similarity of action for plain and deformed bars during the earlier stages of loading.

Concrete beams with sheet steel web plates

J. TRUMAN THOMPSON, THOMAS F. HUBBARD, and JOHN N. FEHRER, *Civil Engineering*, Vol. 8, No. 12, Dec. 1938, p. 815. Reviewed by J. R. SHANK

A number of reinforced concrete beams containing sheet steel instead of stirrups for web reinforcement were made up and tested in comparison with similar beams containing stirrups. This sheet steel occupied the region of the beam where stirrups would ordinarily be used. Thicknesses ranged from one-sixteenth to one-sixty-fourth of an inch. Some had the top and bottom edges bent over to form flanges. Others had circular perforations one-quarter and one-half inch diameter closely spaced. In some beams the tension steel was hooked at the ends.

Strain-gage measurements were taken on the sheet metal as well as on the stirrups. Deflections were also measured.

Stresses and deflections as measured closely corresponded to those computed by ordinary theory up to the cracking of the concrete. The web plates started taking their share of the load immediately and advanced consistently which was not the case for the stirrups. The deflections up to working loads were the same for both types of beams. The ultimate strengths were not markedly different. The thin plates were as good as the thick ones. The flanging did not seem to help nor did the perforations. The patterns of the cracks as the tests advanced were much the same in all cases. A general conclusion is that any thin sheet that can be efficiently handled in construction will serve, which means considerable saving in construction cost and some saving in steel. The weakening of the beam at the stirrup point is obviated as well as the danger of stirrups being too close to the outside of the concrete for safe weathering.

Progress report on warping of concrete pavements

HAROLD ALLEN, (U. S. Bureau of Public Roads) Highway Research Board paper. HIGHWAY RESEARCH ABSTRACT

This report is a record of studies made to find the cause, means of prevention, and correction of warping of concrete pavement. The data were obtained from observations of pavements and from field and laboratory tests in California, Kansas, Missouri, Minnesota and Texas.

The results of a portion of the laboratory tests made as a part of this study to determine the expansive properties of soils were published in the Sixteenth Proceedings of the Highway Research Board.

Cause—The data presented indicate that warping is caused by the differential swell, or shrinkage of subgrade soil. Soil expansion results from absorption of water entering through leaky joints, cracks and at pavement edges. In Minnesota the swell usually develops from the formation of ice lenses, but in California, Kansas, Missouri and Texas, frost action has only a minor effect and the volume change occurs largely because of the expansive characteristics of the soil itself.

The amount of swell is dependent on the characteristics of the soil, on its moisture content and density at the time the pavement is placed, and on the subsequent absorption of moisture.

Means of Prevention—The data indicate warping can be prevented by avoiding expansive subgrade soils. Objectionable warping can be prevented by controlling moisture differential and resultant volume change of the soil. Bases and joint drains were only partially effective in preventing warping. The recurrence of seasonal warping was prevented by efficient joint and crack maintenance.

Means of Correction—The most practical and economical means of correcting objectionable roughness due to warping is by the mud-jack.

Effect of impact on reinforced concrete beams

T. D. MYLREA (University of Delaware), Highway Research Board paper.
HIGHWAY RESEARCH ABSTRACTS

One of the questions which has always been raised whenever the use of high yield point steel has been proposed, is that of its resistance to impact. While experimental studies have been made concerning the behavior of isolated specimens of steel under impact, very little has been done in studying its behavior when used as reinforcement.

It has at times been suggested that the Izod tension impact test should be a reliable guide as to the shock resistance of the various grades of steel reinforcement. It has also been suggested that the area under the tension stress-strain curve could well serve a similar purpose. The tests conducted during the last two years at the university of Delaware have had a double objective: (1) to ascertain the reliability of either of these methods as a criterion for the relative shock resistance of steels of structural, rail, and high elastic limit grades when used as reinforcement, and (2) to determine, by direct test of reinforced concrete beams, the relative merits of these steels when subjected to shock while serving as reinforcement.

It was found impossible to break the steel in the beams reinforced with either one ½-in. or one ⅝-in. round bar. Repeated blows from the full height available, about 8 ft. 4 in., caused increasing deflection and shattering of the concrete, and if continued long enough would rip the bar out of the beam. This indicates very clearly the remarkable insurance against collapse provided by even small amounts of reinforcing steel.

Another surprising phenomenon was the fact that within the limits of the tests no noticeable difference in impact resistance was displayed by any of the grades of steel used, an almost identical drop being required for all grades. The beams reinforced with structural steel deflected more than the others, probably thus absorbing the energy that the beams reinforced with stronger steels withstood by direct stress. The few shallow beams tested deflected so much that in spite of their less mass it was quite as impossible to break the ½-in. round bars in them as it was in the deeper specimens.

As far as the Izod tests are concerned, there is no apparent relation between the strength of the Izod tension impact specimen and that of the same steel when used as concrete reinforcement.

Concrete for Kanawha River project

JOHN C. SPRAGUE, *Civil Engineering*, Vol. 8, No. 12, Dec., 1938, p. 797. Reviewed by J. R. SHANK

Four new dams, one on the Ohio River at Gallipolis and three on the Kanawha River at Winfield, Marmet, and London were built to replace 13 smaller ones. These four contained a total of 820,000 cu. yd. of concrete.

Special cements were specified for the locks and for the dams at the Gallipolis and Winfield sites. For the locks, a minimum of 15 per cent retained on the No. 200 sieve was required, and for the dams, maximums of 55 and 8 per cent respectively, for the tri-calcium silicate and tri-calcium aluminate constituents. These stipulations were made with a view to reducing heat generation. The cements used at the dams had on an average 5 per cent, and the locks 17.5 per cent retained and the average combined percentage of the tricalcium silicates and tri-calcium aluminates was 13 per cent lower in the dams. The tensile strength tests showed that the lower tri-calcium salts percentages just about balanced the greater fineness. Five sacks of cement were used for the concrete for the dams and 4.5 for the locks. Since the cement for the locks was coarser a still smaller effective amount of cement went into

the concrete, if that retained on the No. 200 sieve may be said to be inert. The effective cement factors for the two were then 3.7 sacks for the locks and 4.75 for the dams.

Temperatures were taken in holes from 5 to 10 feet deep. The concrete of the dams showed only a 3 degree F. greater rise in temperature than that of the locks. The effects of the greater fineness predominated slightly over that of the reduction of the tricalcium constituents. It took 5 hours longer for the concrete of the locks to reach its maximum. The rate per hour rise for the dams was greater by 0.18° F.

The compressive strengths of the concretes ranged from 3800 to 5000 p.s.i. at 28 days, the concrete from the special cements in the dams being the stronger after about 23 days. At one year these special cement concretes attained over 6400 p.s.i. Tests of cylinders cored from the works showed that the strength increased progressively with depth up to 5 feet. At 3.8 feet the strengths were the same as for the poured test cylinders.

In the construction, electric vibrators were used at from 3600 to 7200 r.p.m. The higher the rate, the better the results. A continuous spray of water for from 10 to 14 days was used to assist in the curing. When the ambient temperature dropped under 35° F. the mixing water was heated to 150° F. which served to produce the temperatures of the placed concrete at from 50 to 100° F. This was maintained for 5 days after the pouring by means of steam pipes under tarpaulins.

The chemical reactions during hardening of hydraulic cements with especial consideration for retarders and accelerators

L. Forsen. *Zement*, Vol. 27, Nos. 46, 47, 48, Nov. 17, 24, Dec. 1, 1938, pp. 719-725, 737-743, 753-758.
Reviewed by L. T. Brownmiller

Many experimental data are presented in this paper which should be consulted in the original by anyone particularly interested in the subject. The data are presented by the author in a series of concise tables and graphs.

Most of the paper is concerned with the solubility of cement compounds and cements, not only in water but also in lime water, calcium chloride, calcium sulphate, potash and soda solutions. Analyses were made of the solutions at various ages. Time of set, heat of hydration and strength of cements were determined not only of normal cements, but also of cements to which a variety of retarders and accelerators had been added,

The author presents the following conclusions of his work:

1. The cement compounds go into solution in the same stoichiometrical proportions as are found in the anhydrous compounds.

2. The hardening of cements is caused by the formation of $Ca_2SiO_4 \cdot 4H_2O$ from supersaturated solutions.

3. On normal setting the solution contains a large amount of lime hydrate which decreases the solubility of the aluminates.

4. When quick setting occurs the solution contains only small amounts of lime hydrate, so that the aluminates and silicates go into solution rapidly. A gel is precipitated whose low strength is a mechanical property of the gel.

5. Retarders are compounds which reduce the solubility of the aluminates and form a semi-permeable membrane around the cement grains.

6. Accelerators facilitate the solution of the silicates by decreasing the pH and the rate of precipitation of the silicate hydrates by increasing the calcium in concentration, without dissolving the aluminates.

7. Destructive agencies are materials as sugar, borax, humus, which dissolve the aluminates and cause rapid set; or are such compounds as phosphates or fluorides which form an insoluble film around the cement particles.

8. The alkalies, which are present in cement as readily soluble materials, cause quick setting and lower the strength. The action of the alkalies can be decreased by the addition of calcium and magnesium salts whereby they are neutralized to the corresponding alkali salts.

9. Basic slags react like portland cement if the solubility of the aluminates is reduced by the action of a retarder. In the same way the reaction of the silicates can be hastened by suitable accelerators.

Research and experience on erected reinforced concrete structures in Switzerland, 1924-1937

M. Ros, *Eidgenossische Materialprufungs—und Versuchsanstalt fur Industrie, Bauwesen und Gewerbe*, Zurich, Report No. 99, pp. 1-405, 1937 Reviewed by F. L. EHASZ

Exhaustive, carefully conceived observations and tests made in the years 1924-1937 on 50 erected Swiss reinforced concrete structures—26 arch bridges, 9 girder bridges and 15 buildings, towers, and frames are reported in a very attractive volume. It is pointed out that experience must be conceded at least the same weight as theories, which depend upon technical research and scientific fundamentals. Actual behavior of structures is to be ascertained only through practical research and experiment on erected structures and it is important to bring general considerations, calculation, construction, materials testing in the laboratory, practical research, constant control at the building site, and experience on completed structures into very close, mutual relationship.

Many conclusions are drawn from this extensive investigation. Experience justifies the statement that concrete structures may be designed and constructed according to the theory of elasticity, in spite of the special properties that are inherent in concrete, such as heterogeneity, anistropy, shrinkage, intricate behavior with the variable factors, loading, number of stress cycles, temperature, time, ingredients, and methods of production, placing and curing. The allowable stresses σ_{ul} are generally between 0.4 and 0.5 of the prism compressive strength ${}_p\beta_d$. The much discussed $n-$ question ($n = E_s/E_b$) is of no importance with regards to magnitude within certain ranges, provided the allowable stresses are synchronized to the chosen $n-$ values ($n = 10$, 15 or 20). Recent agitation for complete omission of the $n-$ ratio is to be discredited, since complication and confusion would result rather than simplification and clarity. The ratio is felt to be very serviceable and basically indispensable. Static and fatigue failures, from which the factors of safety were obtained are not dependent upon the $n-$ ratio, but on some of the factors, compressive strength of concrete ${}_p\beta_d$, yield point of reinforcing steel σ_s, fatigue strength—original or virgin strength (Ursprungsfestigkeit)—of concrete ${}_b\sigma_u$ as well as of steel ${}_s\sigma_u$, the percentage of steel μ, and the shape and dimensions of the cross-section. Average values of the factor of safety ν_r at the age of 90 days are given in tabulated form:

	VALUES OF ν_r		
	Static Failure (all influences)		Fatigue
	Without Temperature and Shrinkage	With Temperature and Shrinkage	
Compression	4	3	1.8
Bending			
Normally reinforced	2.0	1.8	1.8
Over-reinforced	2.5	2.2	2.0

Buckling			
Column longitudinally or spirally reinforced....	4	3	...

The difference between the actual and calculated factors of safety depends on: the extent of agreement between the assumptions of the static or dynamic calculations and the actual conditions; constructional details; strength and other qualities of the materials used; sharpness of control during construction; and the efficiency of the construction itself. An estimate of the actual safety, therefore, had to be determined in each particular case. Variations which arise in the observed states of stress, deformation, failure, fatigue or buckling compared with the normal stress distribution, deformation, etc., are to be considered as far as possible in the estimation of the effective factor of safety. Such variations appear especially in highly indeterminate structures (frames, slabs, arches, continuous beams, etc.). Expensive failure tests of prototypes or models of structures, in which the most unfavorably stressed section breaks only and for an entirely fixed case of loading, give very valuable information, but can, as a rule, be dispensed with. They are believed not to contribute more than practical tests and experience on completed, standing structures, in conjunction with materials testing in the laboratory and control at the building site. To the same extent as the study of materials under uni-, bi-, or tri-axial states of stress, especially in present times of efforts for very high allowable stresses, there has become indispensable the important consideration of fatigue. For concrete in compression the virgin strength is reported to be approximately 0.6 of the prism compressive strength ${}_p\beta_d$. In the case of reinforcing steel in tension the virgin strength is approximately 0.85 of the yield point σ_s. Results of strength and elasticity tests in the laboratory are in very satisfactory agreement with results of measurements on erected structures. Initial hardening of the concrete produced on the building site depends on physical and colloidochemical characteristics which are very sensitive compared with all external influences. The strength and elastic properties of the concrete depend on the quality or brand of cement, the amount of cement, grain-size distribution of the sand and gravel, the amount of mixing water (consistency), the effectiveness and manner of production, the temperature, weather circumstances, curing and age. Each of the quoted factors is of a more or less varying nature. Combinations of these factors serve to produce fluctuations in the strength and elastic properties. Proper conditions and control at building sites have been shown to restrict the variations to within $\pm$ 20 per cent up to $\pm$ 25 per cent of the average values. Properly designed and erected reinforced concrete structures showed very good agreement of stresses and deformations obtained by calculation and measurement, departures of over 25 per cent being attributable to disturbing influences produced by the structure itself (roadway and frictional resistance, expansion joints, shrinkage cracks) or by outside influence (temperature and time).

Between the modulus of elasticity E_e and the prism compressive strength ${}_p\beta_d$, there obtain for compressive stresses up to $\sigma \leqq 0.33\ {}_p\beta_d$ the relations

$$E_e = 550{,}000\ \frac{{}_p\beta_d}{150 + {}_p\beta_d} \text{ for concrete}$$

and

$$E_e = 600{,}000\ \frac{{}_p\beta_d}{300 + {}_p\beta_d} \text{ for mortar,}$$

expressed in kg./cm^2. Knowledge of the average modulus of elasticity E_e of concrete or reinforced concrete structures makes it possible on the basis of its elastic behavior

to arrive at the average concrete strength and thence at the safety of the structure with an accuracy sufficient for the purposes of practice. These relations, formulated from laboratory tests of the E. M. P. A., were found to be quite reliable. The modulus of elasticity E_e is said to relate exclusively to the practical elastic strains. For the total strains the continually smaller deformation modulus V_t (Verformungsmodul) is decisive. In examining test results due attention must be given to these properties: E_e is to be considered for the ordinary course of loading and V_t for other cases such as are involved in the removal of scaffolding. Older concrete of high strength exhibits by far smaller flow characteristics than fresher concrete of lower strength. In the course of two years the amount of flow attains on the average roughly 40 per cent of the total strain corresponding to that of the load and shrinkage factors. Concrete which has attained its final strength does not flow appreciably and is resistant against frost, corrosion and weathering. The determination of calculated values of stresses and strains in reinforced concrete structures, made for purposes of comparison with measured values, had to follow on the assumption of operative action of the entire concrete section, including the tension areas and with $n = E_e/E_b$ varying from 10 to 4 and on the average equal to 6, depending on the quality of concrete. Only exceptionally, in the checking of strains, is the reduction of moment of inertia due to fine cracking in tension zones to be taken into account. Impact numbers for arch and girder bridges were found to lie within the range prescribed by Art. 16 of the Eidg. Eisenbetonverordnung of May 14, 1935. $\varphi_3 = \dfrac{550 + 5l}{10 + l}$, expressed in percentages of the static loads, where l is the span or half the arch span, as the case may be, in meters.

Results obtained from tests on highly indeterminate systems, such as flat slabs in buildings, are to be attributed equal or even greater value than results of theory whenever the theory contains simplifying assumptions which may not be entirely justified. Careful strain measurements during the gradual, deliberate unscaffolding process guard against overloading and damage. Fine crack formation has no deleterious effect on the stability of reinforced concrete structures, if the production and curing operations are properly carried out. Large-span arch bridges, slender columns and thin slab and shell structures require especial considerations of buckling. Properly designed and carefully erected reinforced concrete structures suffer no destruc tive effects due to the action of unpropitious temperature and weather influences; yearly temperature fluctuations between $+25°$ and $-20°$ C. and daily temperature differences of 15° C. are noted. The absence of danger of rusting and the little or no maintenance in reinforced concrete structures are touched upon.

A large number of excellent photographs, graphs and diagrams is included in this well-prepared, highly commendable report. The Swiss investigation should serve as an impetus to similar research in other countries.

Vol. 35 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

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AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN *APRIL 1939*

The President's Address*

By John J. Earley

It is customary in the American Concrete Institute for the retiring president to address the members in convention and to choose his own subject. I, therefore; have this to say. Last year when you elected me to be president I pointed out to you that you had broken precedent in choosing a craftsman. That I would naturally see things from that point of view and that I would try to interest other craftsmen in the Institute. After a year in office I wish to supplement that statement and to speak to you about craftsmen in the Institute.

When a man is president he is conscious of the Institute with a consciousness not known in any other office. It is as if a window had been opened to him through which he sees a more complete and more extended Institute. What I have seen has given me the impression that the American Concrete Institute is becoming more academic in character than its charter permits. I feel that it is not properly disseminating the knowledge acquired, which is one of its vital functions. When the vital functions of the Institute are well balanced it is in good health, but when they are unbalanced the Institute is ill. And I feel that the function of dissemination is not good and I fear that unless a suitable remedy be applied atrophy will slowly creep through the whole organism.

The American Concrete Institute is a corporation created by a legal convention and given certain powers or functions more or less closely defined by a charter. The charter of the American Concrete Institute says that the particular business and objects of the corporation shall be to disseminate information and experience upon and to promote the best methods to be employed in the various uses of cement

*Thursday, Mar. 2, 1939, at the 35th Annual Convention, American Concrete Institute.

by means of convention, the reading and discussion of papers upon materials of a cement nature and their uses, by social and friendly intercourse at such conventions, the exhibition and study of materials, machinery and methods and to circulate among its members by means of publication the information thus obtained.

I have long been convinced that the American Concrete Institute by the normal exercise of its functions has obtained the greatest single store of knowledge and experience upon concrete which now exists. This is the talent of the Institute. It is recognized by the concrete industry of the world; and on it rests the prestige of the Institute. This knowledge and experience is made available to all members of the Institute and is to them a source of power and a mark of distinction. They are known and respected and employed because of this knowledge. Many have received national or even international recognition. Around such men are found other men hungry for knowledge and distinction. Around these men are other men hoping to find what will help them in their business. This is the vital function of the Institute, to gather the talents of all and distribute them to each and the experience of each and to distribute it to all.

Ills may befall any creature even if it be a legal entity. Any disorder occurring in its constitution or its function may be regarded as an illness. For example, if the Institute gathered knowledge without experience by greatly increasing the number of scientists and decreasing the number of craftsmen, it might become a purely scientific society and change the norm of its own existence. On the other hand the number of craftsmen could be increased and the number of scientists decreased until the society became a craft guild. In either case the society would no longer be the American Concrete Institute. Again, if the American Concrete Institute developed a mode of dissemination of knowledge and experience on concrete which was a precisely correct expression of scientific knowledge but which was unintelligble to its craftsmen; or if the Institute developed an impressionistic expression of experience on making concrete, which was without reasonable foundation to its scientists then the Institute's channels of dissemination would have become clogged, a function of the Institute would have been impaired and the Institute would be sick.

As a member of the American Concrete Institute for about twenty years, and an officer, off and on, for nine years, I have become conscious, very conscious, of a tendency in the operations of the Institute to accent the accumulation of scientific knowledge and to slur the accumulation of practical experience. I am also conscious that the expression

of scientific knowledge has been perfected in form until it is no longer intelligible in the arts. As a result of this the American Concrete Institute is in my opinion slowly but surely changing its character towards that of a close society of professional and scientific men interested in the nature of portland cement, its relations to other materials and the design of plain and reinforced concrete structures. The work which this group of men has been doing is of the best, it's value cannot be denied, too much credit cannot be given to them. The results of their work have been incorporated into the records of the Institute where they may rest without value, to those, who make concrete until some one with leisure and ability and what Harvey Whipple calls "the urge" reforms them for use in the arts.

Perhaps it is too much to expect that someone will undertake to recast the great accumulation of matter in the records of the Institute but surely it is not too much to expect that the Institute, assuming that it does not wish to change its constitution to that of a purely scientific society, will cause its present and future scientific accumulations to be both *precisely* expressed for the scientific group and *descriptively* expressed for the use of craftsmen. It is also to be hoped that descriptions of experiences in the field will be gathered and suggestively related to scientific principles for the benefit of the scientific group.

In a society such as the American Concrete Institute, it is not possible to cure in one year maladies which have already become chronic but it is possible to point out and to make a beginning of the necessary corrections. It has not been difficult to direct the attention of the officers of the Institute to the symptoms. It is not difficult for you to see them. Look about this convention and ask yourselves where are the manufacturers of cast stone, where are the makers of concrete blocks and pipe, how many here run mixing plants? And where are the field superintendents of contractors and foremen under them? All such people need the knowledge of how to make good concrete and it is a function proper to the American Concrete Institute to give it to them.

The officers of the Institute have begun to re-establish proper balance in the functions of the Institute by means of its technical committees. A new technical committee, 412, has been established. It has been given one of the most difficult assignments ever given to an Institute committee. It has been instructed to review all that is known in its field, and to produce a specification and recommended practice which will be equally intelligible and useful in science, in art,

and in industry. The report of this committee is looked for hopefully by the officers of the Institute. If it be satisfactory the organization and operation of other technical committees will be patterned after it, in order that the work of the American Concrete Institute may be extended in an ever widening circle, which will include owners, architects, field engineers, foremen and craftsmen who are the makers of concrete.

I think this is all that I wish to say. We will have no exhortations to make. The matter is before you for your information and what you will do about it is in the hands of future administrations.

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The Navy's New Ship Model Testing Plant*

HUGO C. FISCHER†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

This paper describes certain features of the concrete construction of the new U. S. Naval Model Basin at Carderock, Md., in particular:

1. The procedure for the manufacture and placing of the precast exposed aggregate panels used as the face forms for the concrete walls of the shop, office, laboratory, power plant and turning basin buildings.

2. The reasons for the adoption of a three-hinged concrete barrel arch for the roof of the main basin building and the construction procedure for placing the reinforcing and concrete of the arch, including the setting of the steel for the Mesnager type hinges used.

3. The procedure for minimizing future movement of the basin walls on account of shrinkage, changes in moisture content and live load deformations.

4. The "plastic mosaic" decorative concrete used for the walls of the entrance lobby and museum in the office building.

THE function of a ship model testing plant, reduced to its simplest terms, is to determine the effects of variations in hull shapes on the power required to obtain given speeds. The improvement of hull design is of the greatest importance since, in a man-of-war, every pound that can be saved in the weight of propelling equipment and fuel for a given speed and cruising radius is available for armor and armament. In a merchant ship, the saving in operating cost for fuel may be decisive in a highly competitive field.

The facilities required for a complete establishment of this nature include:

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†Commander, Civil Engineer Corps, United States Navy—officer in charge of construction, U. S. Naval Experimental Model Basin, Carderock, Md.

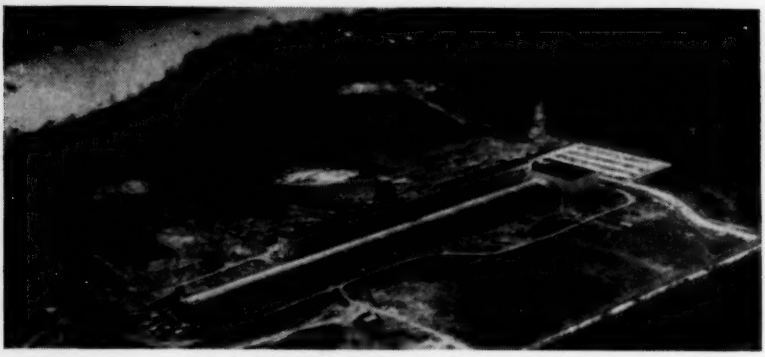

FIG. 1—AERIAL VIEW OF SHIP MODEL TESTING BASIN, CARDEROCK, MD.

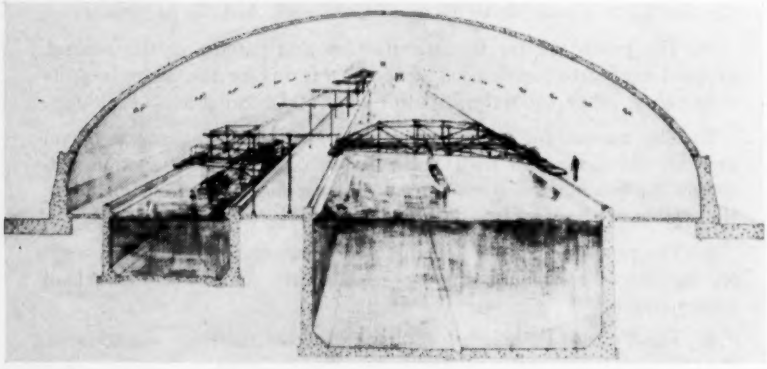

FIG. 2—CROSS SECTIONAL VIEW OF SHIP MODEL TESTING BASIN, U. S. NAVY

OFFICIAL UNITED STATES NAVY PHOTOGRAPHS

(a) a deep basin for tests of seagoing ship models; (b) a shallow basin for tests of river boat and tug models; (c) a turning basin for tests of rudder action; (d) a high speed basin for testing models of motor boats, high speed craft and friction forms; (e) a dry kiln for conditioning the lumber used for models; (f) a wood working shop for the manufacture of the models to be towed and self propelled in the basins; (g) a paint shop for painting models; (h) a fitting out room where the models are trimmed and adjusted and fitted with propellers, etc.; (i) a machine and metal working shop for the manufacture of model propellers, construction of models used to study the elastic properties of ship structures and manufacture of dynamometers and special instruments used in model basin research; (j) a hydraulic laboratory including variable pressure tunnels for testing propellers and a small model basin for the testing of special forms, for preliminary tests with small models on unusual research problems, and for miscellaneous research in hydro-dynamics; (k) a structural materials laboratory for testing of structural models up to 30 and 40 feet in length

and 6 to 8 feet in width including equipment for research on vibration of ship structures and similar investigations; (l) a complete photographic laboratory including all facilities for reproduction of photographs and drawings; (m) drafting and computing rooms for the design of models, apparatus and evaluation of results of tests; (n) offices for administrative personnel; (o) heating, power and water services.

The ship model testing plant now being built for the Navy by the Turner Construction Co. at Carderock, Md., will replace an old plant at the Washington Navy Yard constructed about forty years ago and now inadequate, obsolete, and structurally unsatisfactory. A brief description of the proposed new plant was given in a paper entitled "Articulations for Concrete Structures; the Mesnager Hinge" by Commander (now Rear Admiral) Ben Moreell, in 1935.* Admiral Moreell was in direct charge of and is responsible for the design of this project. The construction is now being carried out under his direction as Chief of the Bureau of Yards and Docks and a more detailed account of the design details and construction operations can be given, with special reference to some of the unusual requirements and how they were met.

UNUSUAL REQUIREMENTS

Small tolerances permitted in construction

The basin walls were allowed a tolerance of one quarter inch plus or minus in alignment as constructed but must have no appreciable subsequent movement since they serve to carry the rails for the towing carriages. These rails are of special section weighing approximately 165 lb. per yd. They are bolted solidly to a series of heavy cast iron chairs which are in turn bolted and grouted to the tops of the concrete basin walls. The heads of the rails are machined and ground and are to be set and maintained to a tolerance of .005 in. in line and in grade parallel to the earth's surface. These tolerances appear fantastic, until consideration is given to the degree of magnification in transferring the results from a standard 20-ft. model to a full size ship; then the concern of the operators for the utmost precision of equipment may be appreciated.

Low leakage from the basins

The cost of raw water from the Great Falls of the Potomac delivered through the Washington Aqueduct by gravity is small. However, it is desired to avoid the necessity for taking large amounts of make-up water at times when the Potomac may be carrying a heavy silt content thus muddying the water in the basins and also to avoid setting up currents from the inflow of considerable quantities of make-up water.

*JOURNAL, Amer. Concrete Inst., Mar.-Apr.-1935; *Proceedings* Vol. 31, p. 368.

Temperature and light control

Constant temperature conditions are desired in the Basin building for accuracy of results in running models. Uniform lighting conditions are desired for photographic work which can best be obtained by excluding natural sunlight. This also serves the purpose of retarding marine growths. To facilitate these requirements the Basin building is constructed without windows or skylights.

Architectural appearance

The establishment is in the National Capital area and faces the projected extension of the George Washington Memorial Parkway. An appropriate exterior is highly desirable.

STRUCTURES REQUIRED

The buildings and structures provided to meet the foregoing requirements are all of reinforced concrete on bedrock foundations and include:

A Basin Building

The Basin building is 1300 ft. in overall length consisting of a main section 1188 ft. long with a clear inside width of 112 ft. meeting a rectangular section 150 ft. wide by 112 ft. long housing the turning basin which is a "J" shaped continuation of the shallow basin. The arch roofed main section incloses the shallow basin 303 ft. long 51 ft. wide with water depth which can be varied from 10 ft. to a few inches, a deep water basin 963 ft. long, 51 ft. wide with 22 ft. water depth, and a high speed basin 1168 ft. long, 21 ft. wide with 10 ft. water depth. The shallow basin is a straight continuation of the deep basin and by removing the floating caisson which separates the two, the accelerating portion of the run can be started at the shallow basin end making a total length of run of more than 1200 ft. The proportions of the basins are considered adequate for the use of considerably larger than the standard 20 ft. length of models, if desired for special work, without measurable side or bottom interference.

Practically all interior surfaces of the basin building are lined with a 2-in. thickness of Thermax for heat insulation and sound absorption and as previously indicated, there are no windows or skylights for natural light or ventilation.

Shop, Office and *Laboratory*

Buildings for the shops, offices and laboratories constitute a continuous group 871 ft. long, and 54 ft. wide, parallel to the basin building and connected thereto by a fitting-out room 78 by 48 ft.

The shop building is 350 ft. long and includes a wood working shop on the first floor, and a group of metal and machine shops on the mezzanine and second floors.

The laboratory building, 330 ft. long, will house variable pressure tunnels for testing propellers, testing machines for structural models, including a 4,000,000-lb. universal testing machine which is considerably larger than now is available in the vicinity of Washington, and equipment for research on vibration of ship structures. There is also included in the basement of this building a small model basin 142 ft. long, 10 ft. wide with 5½ ft. water depth.

The office building, three stories high with a tower inclosing the potable water storage tank, is located between the two story shop and laboratory buildings and forms the central architectural feature of this group. It will contain in addition to the usual administrative facilities the offices of the commissioned and civilian technical personnel, the drafting and computing rooms, a reference library, several record storage vaults, an unusually complete photographic laboratory and a museum.

A heating plant, electrical substation, waterpurification plant, sewage disposal plant and the usual roads, streetlights, fire protection system, etc. complete the establishment.

The remainder of this paper will be devoted to special features of design and construction.

PLASTIC MOSAIC

The walls of the main entrance lobby and walls and ceiling of the museum in the office building are finished in plastic mosaics by John J. Earley, who originated and developed this type of decorative concrete. Wall panels and pilasters are being precast at his studio and ceiling areas are being applied in place directly on the structural concrete. The precast panels are bonded securely to the concrete and tile walls by means of metal ties and grout filler.

The basic field color is a light buff with colored mosaic designs for borders, soffits and trim of openings. As the high decorative feature there are six ship panels, three each across the east and west walls of the museum, each of the six selected ships typifying definite stages of progress in propulsion and hull design of American vessels along the lines of the research and experimental work to be carried on by this establishment.

The process of manufacture of plastic mosaic may be briefly described as the placing of a facing of portland cement concrete made with aggregate of the desired colors in a specially made mold, backing the colored aggregate facing with plain aggregate concrete to the desired thickness for handling and setting, removing the panel from the mold as soon as it has the necessary strength for handling, sanding

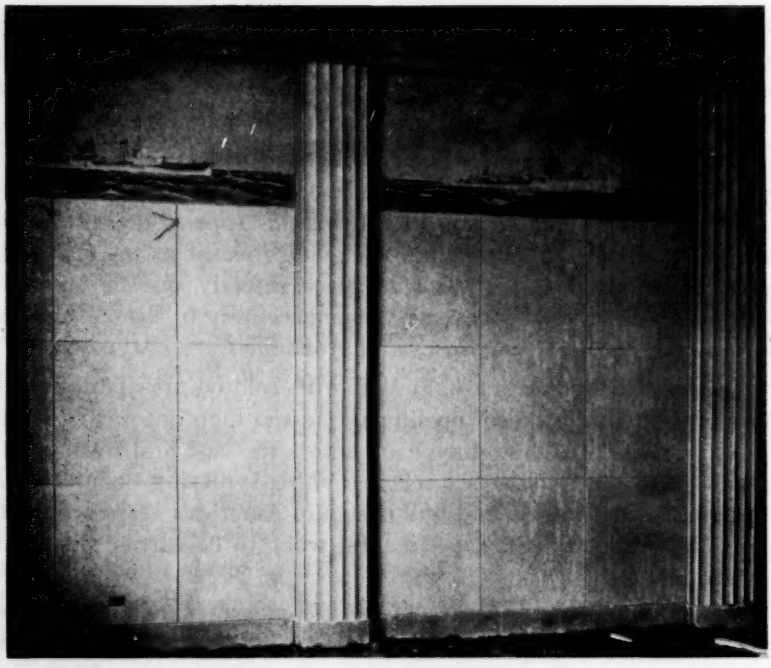

FIG. 3 AND 4—OFFICE BUILDING, MUSEUM, SHOWING INSTALLATION OF PRECAST PLASTIC MOSAIC PANELS

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and brushing the face as the first step in cleaning off excess mortar and then acid washing the face until the aggregate is fully revealed and the colors of the design well brought out.

So described the process appears simple and the result also has the merit of simple beauty. Upon closer examination, however, one finds that the achievement of this result was far from simple, that the simplification of the design of a full rigged ship to meet the requirements of the medium and yet show even the cordage against the background of the sky, the selection of colors to produce the proper effect of light and shade and the sea, and the craftsmanship involved in carrying out the designs, all require the work of a genuine artist and an organization with years of experience and research extending even to the production of the aggregates of mosaic glass and vitreous ceramics for colors which are not available in natural stone. Plastic mosaic is essentially a studio product and will no doubt so remain, its future depending on the artistry and experience of the producer.

PRECAST EXPOSED-AGGREGATE FACING

The exterior surfaces of the shop, office and laboratory group of buildings, the fitting out room, the turning basin building and the boiler house and substation buildings are faced with precast exposed-aggregate reinforced-concrete panels which served as the outside form against which the structural concrete was cast. This type of exposed aggregate concrete was also originated and developed by Mr. Earley and he was retained as expert consultant for the work. However the actual production of the panels for this project, totaling some 135,000 sq. ft., was undertaken by The Dextone Co., New Haven, Ct., with no previous experience in this type of facing. This material is believed to have a wide field of usefulness in that the cost is less than natural stone facing; it can be cast in much larger sections than practicable for thin slabs of natural stone, thus facilitating watertightness and allowing the jointing to be placed where desired for architecture effect; and by a selection of aggregates it can either be given color or, as on this project using quartz, a higher light reflectance, thus adding life to large unfenestrated areas.

The panels are 2 in. thick, and of variable size up to 8 ft. wide by 10 ft. high. The facing mix is 1 in. thick and consists of 1 bag of white cement (94 lb.) to 90 lb. of fine aggregate and 300 lb. coarse aggregate. The coarse aggregate consisted of 1 part translucent and 2 parts opaque quartz all but a fractional per cent passing a $\frac{3}{8}$ in. sieve and being retained on a No. 8 sieve. The fine aggregate consisted of crushings from both opaque and translucent quartz all

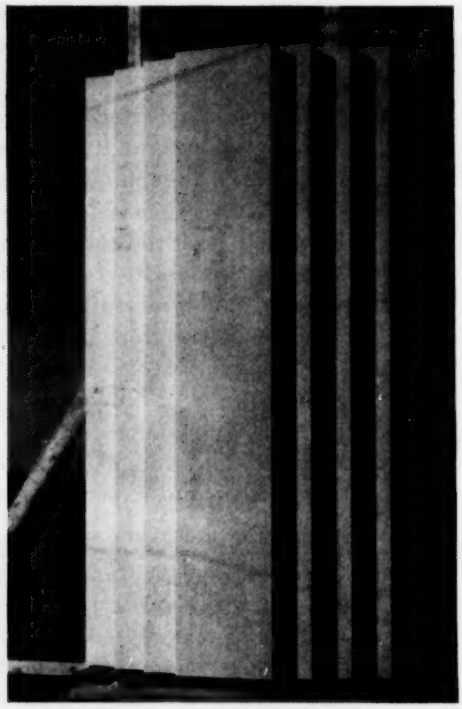

FIG. 5—FULL SIZE SAMPLE PILASTER OF PRECAST EXPOSED AGGREGATE PANEL

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passing a No. 16 sieve and being retained on No. 100. The backing mix also 1 in. thick consisted of 1 bag portland cement to 90 lb. bank sand and 306 lb. trap rock.

A water cement ratio of 6 gal. per bag of cement was used on the facing mix except for returns where it was reduced slightly to prevent sand pockets. No account was taken of surface water in the aggregate as the facing materials were kept in dry storage. For the backing mix of which the aggregates had a variable water content as delivered in trucks, the added water varied between 4 and 5 gals. per bag of cement as required to produce a fairly dry mix. The resulting concrete averaged a 28-day compressive strength (on 2-in. cubes) of 7525 p.s.i.

Molds were constructed of wood, shellacked and then lined with masonite. The masonite lining was given two coats of shellac, then a coat of caster oil (1/3) and shellac (2/3). Before casting, the forms

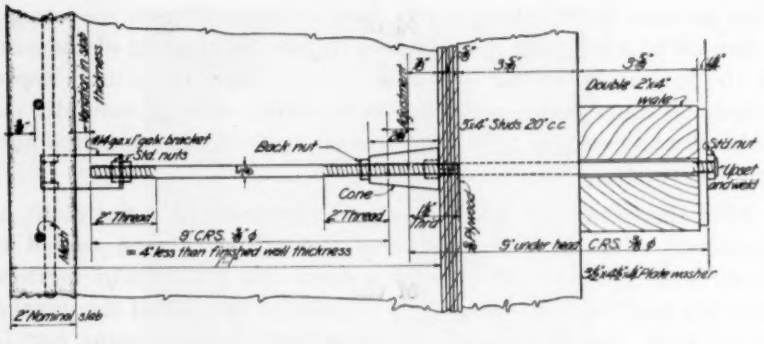
were greased with animal fat (tallow or lard oil) and soapstone, (1 gallon oil to 5 lb. ground soapstone) to prevent sticking of the casting to the mold. With this treatment forms could be re-used approximately 60 times before relining was required. Strong, carefully made molds are an essential to produce the sharp true edges required and to avoid warpage.

After the facing mix was placed and vibrated with a flat plate, high frequency vibrator, the reinforcing, consisting of zinc coated cold drawn wire .3065 in. in diameter welded into a 4 by 4 in. mesh, was placed in position not less than $\frac{3}{4}$ in. back of the face of the panel and the backing mix then placed and vibrated. Returns being cast in a vertical position were vibrated internally with a half inch rod pushed into the concrete. The backs of the panels were gone over with a stiff fiber brush several hours after casting to roughen for bond. Anchor loops consisting of 1 in. wide by $\frac{3}{32}$ inch thick galvanized strap iron with the ends hooked around the reinforcing mesh and the loop part projecting about an inch clear of the concrete, were also installed on 2 ft. centers each way.

The panels were removed from the molds in from 12 to 24 hours depending on the air temperature, 12 to 14 hours for 80°F. and 18 to 24 hours for 50°F. The lower temperature is preferable to avoid excessive shrinkage resulting in hair cracking. As soon as a panel was removed from the mold it was placed on a traveling easel and gone over with an electrically driven belt sander to remove excess mortar on the face and then finished with a wire brush. Each panel was then checked for size, trueness, broken edges, sand pockets, etc. and repaired if necessary. Repairs were made within 36 hours of casting.

The panels were then cured for 3 days, being thoroughly sprayed with water twice a day. On the third day a solution of 1 part of muriatic acid to 5 parts of water was applied and then brushed off using plenty of water. The proper degree of acid etching for uniform texture was then checked by an inspector under a strong light and the panels were cured four more days wetting them twice daily. On the seventh day they were given a final light acid wash (1:7) except for spots that might need a stronger application to bring them to the texture of the standard sample. The panels were then ready for shipment.

Great care is necessary to avoid staining the surface; the masonite form lining must be reshellacked about every third use, only brushes of the non-staining fiber type can be used, acid must be thoroughly washed off and the face must not rest against wood. Celotex or other fiber board wrapped in non-staining paper makes a good cushion



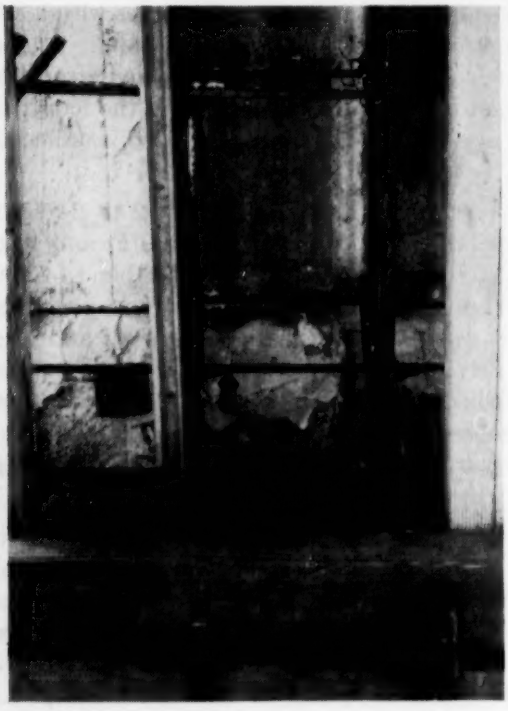


FIG. 6 AND 7—ABOVE, A DETAIL OF THE BOLT ASSEMBLY FOR HOLDING BACK FORM TO FACE OF FORM PANELS. BELOW, A PHOTOGRAPH (INSIDE). LATER ON THE BOLTS WERE SET BETWEEN PANEL JOINTS TO ALLOW POURS UP TO 8 FEET

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between the stones and wood bracing for shipment or handling. While as stated these panels were produced by a manufacturer who had no previous experience in the production of this type of panel, the process is far from simple and considerable time was required for development of facilities and training of personnel before a product of acceptable quality was obtained.

SETTING AND HOLDING THE PRECAST PANELS IN PLACE AS FACE FORMS

The panels were set on mortar beds with quarter inch joints in the manner usual for natural stone. One end of a ⅜ in. bolt was then secured to the anchor straps embedded in the back of the panel with a nut on each side of the strap. A nut which remained in place for mechanical bond and a cone which was later removed were threaded on the other end of the bolt, the cone serving as a spacer for the inside of the back forms. Another bolt was threaded into the base of the cone and passed through wales supporting the ⅝ in. thick plywood back forming and the whole drawn up tight.

It will be noted that the face panels and back forms were secured by lines of bolts at 2 ft. centers and 2 ft. intervals in height which was computed to be adequate for concrete pressures of a 5-ft. height of pour. In actual experience it was found, however, that the joints could not be held to the required accuracy of line unless the first 2 or 3 ft. was poured first and allowed to set to hold the lower meeting edge before pouring the rest of the height of the panel. Later on tie bolts were installed passing through the joints between panels to hold the meeting edges in line, thus allowing pours up to 8 ft. high.

The back forms were removed by backing out the stud bolt from the cone through the wales; the cone was then unscrewed and removed but the bolt from the panel anchor strap to the nut back of the cone remained in place for mechanical bond of the panel to the structural concrete. The holes left by the cones were filled with mortar.

After the exterior of a building was completed, the joints were pointed with a mortar made of white cement and quartz sand, matching the panel surface. Various types of joints were tried but a beaded joint with the convex surface flush with the face of the panels was found most satisfactory in producing clean sharp joint lines. A few minor repairs to edges damaged in handling were also made at the time the joints were pointed. After allowing not less than a week for the pointing mortar and repairs to set, the entire exterior was given a light acid wash thoroughly flushed off with running water.

The material lends itself excellently to bringing out the architectural motif of the group of buildings, which is a clarity of line, thought and purpose, typifying the function of the establishment. This motif was also carried out in the structural design as multi-story rigid frames, taking account of the effect of a load on any part of the structures upon the stresses in all other parts, in one of the earliest applications of modern principles of analysis of continuous frames on buildings of this size and loading. The structures are considered as single units

permitting smaller and more economical sections and resulting in a better balanced design of the entire structure than is the case when the more conventional procedure is followed. The degree of integrality of these panels with the structural concrete was also taken advantage of by allowing the half thickness as part of the structural strength of the buildings.

While the use of these panels as face forms involves considerable difficulty in holding them in line and there is a restriction to progress by having to set each course of panels and pouring the backing concrete before the next increment in height of the building can be undertaken, this method is considered fully justified on a project of the nature of this establishment where the buildings are relatively low and more than half the work does not involve this type of construction. However on tall building projects these elements would no doubt constitute a serious objection by greatly increasing the time required for construction.

For tall buildings it is believed that this material has a large field of usefulness as a facing applied in the same manner as natural stone and backed up with concrete block or brick with the advantage of having fewer joints and less work in setting since the panels can be cast in much larger sections. Entire spandrel or column sections can be cast in one piece or as desired for architectural effect thus also improving watertightness and by the provision of special setting lugs and anchors cast on the backs of the panels they can be set and backed up in less time than natural stone. By this method the structural frame can progress at the rate of a story per week and the closing of the building follow without delay.

THREE-HINGED REINFORCED CONCRETE ARCH

The roof of the basin building is a three-hinged reinforced concrete barrel arch. The span between abutment hinges is 110 ft., with a rise of 24½ ft. The thickness at the crown is only 8 in. and at the abutments 12 in. The roof structure is 1188 ft. long, with contraction joints 60 ft. o.c. and cork expansion joints 120 c.c. There are no structural ribs or projections, the interior presenting an absolutely smooth surface and the exterior having false ribs which serve as waterproofing covers for the contraction and expansion joints.

The considerations which led to the adoption of this unique type of structure are:

(a) Comparative studies indicated that the first cost would be somewhat less than for the conventional type of steel truss design.

A recent estimate based on the actual cost for the arch construction indicates the saving to be about 13.5 per cent.

(b) The arch furnishes a smooth interior surface which permits even distribution of the artificial lighting and eliminates air currents which might result in disturbances of the water surfaces.

(c) The cost of periodic painting of steel work is eliminated.

(d) The function of the establishment being unique, it was considered that a structure whose form was distinctively different from the conventional types would be especially appropriate. With the conventional type of construction in a structure a quarter of a mile long, without doors and windows, it would have been difficult to avoid the appearance of an elongated cold storage building.

The hinges are so-called "Semi-articulations" of reinforced concrete. This articulation was invented by the eminent French engineer, Augustin Mesnager, and has been used extensively in bridge and building construction in Europe.

Because of the unusual character of the arch and the slender dimensions, it was decided to undertake a series of tests of the hinges and a loading test of a quarter scale model of the arch.

The load test of the arch model was described briefly by Admiral Moreell in his paper for the Institute in 1935 (previously referred to) and the hinge tests are described and discussed in papers by Admiral Moreell and by Parsons and Stang.* The tests served to verify the adequacy of the Bureau's designs and to bring this type of hinge to the attention of practicing engineers and research workers.

SETTING ARCH REINFORCING STEEL

The reinforcing steel for these slender arches was required to be placed to unusually close tolerances in order to meet the conditions of the design. The hinge bars and hinge bar stirrups particularly were found not cut and bent with sufficient accuracy under usual commercial practice as received from the mill to obtain true line intersections of the hinge bars and the essential metal to metal bearing of stirrups on bars. After a laborious and expensive attempt to correct the bending of the bars when setting them, wedging the loose stirrups and with much difficulty in holding the intersections in line, the contractor arranged to have the bars welded to the stirrups in units of four at the mill. This solved the difficulty and with the further provision of a jig secured to the substantial abutment forms, with slots into which the bar units were placed and held by a through

*"Tests of Mesnager Hinges," JOURNAL Amer. Concrete Inst., Jan.-Feb. 1935; *Proceedings* Vol. 31, p. 304.

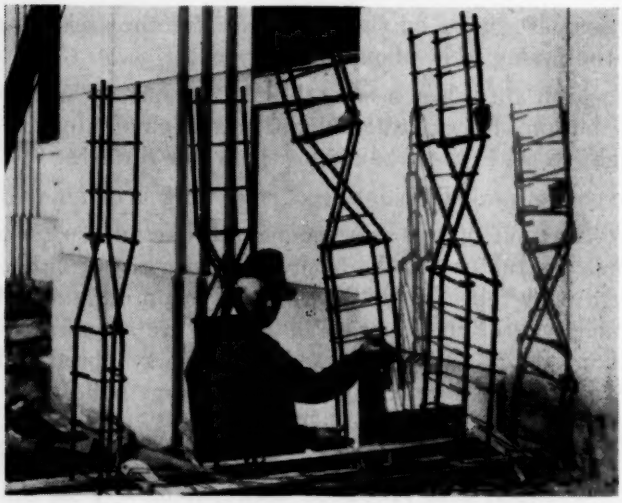

FIG. 8—WELDED HINGE ASSEMBLIES AS RECEIVED, ABUTMENT HINGE UNITS AT RIGHT, CROWN HINGE UNITS AT LEFT

FIG. 9—ARCH TRAVELER SET FOR SIXTH POUR

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bolted cap piece, the hinge bars were set accurately and rapidly at a tonnage cost no greater than for ordinary reinforcing steel set to usual standards.

The main reinforcing bars of the arch are of hard steel in single lengths of 62 ft. for each half of the arch. They were bent to the radii of the arch at the mill and assembled in mats of 10 ft. width on wood forms curved the same as a half arch. These forms were slotted at 6-in.

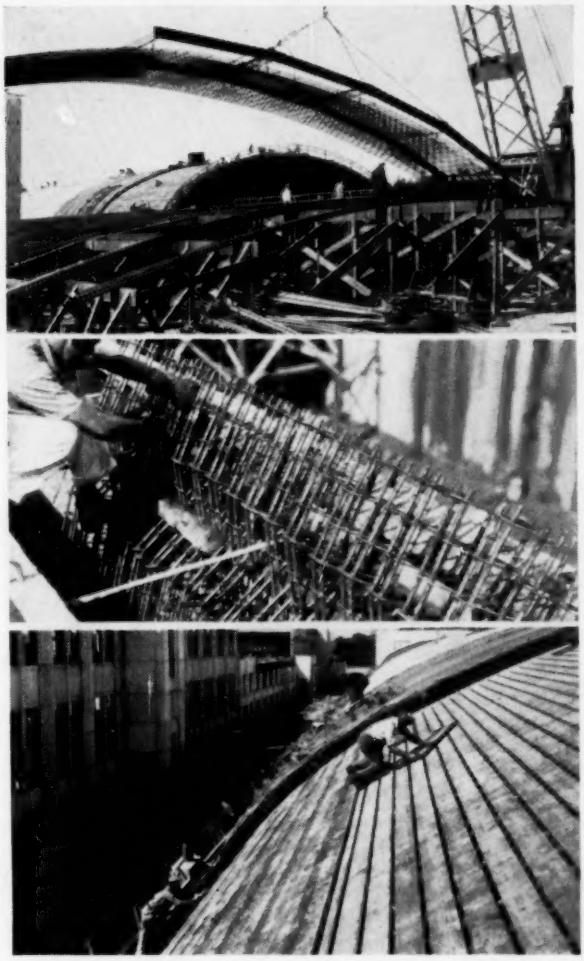

FIG. 10—SECTION OF ARCH REINFORCING STEEL PICKED UP BY TRAVELING CRANE FOR PLACING ON ARCH CENTERING

FIG. 11—SETTING OF ABUTMENT HINGE UNITS, SHOWING CLAMPING JIG USED TO INSURE EXACT SETTING

FIG. 12—PLACING INSULATION SERVING AS BOTTOM FORM OF ARCH

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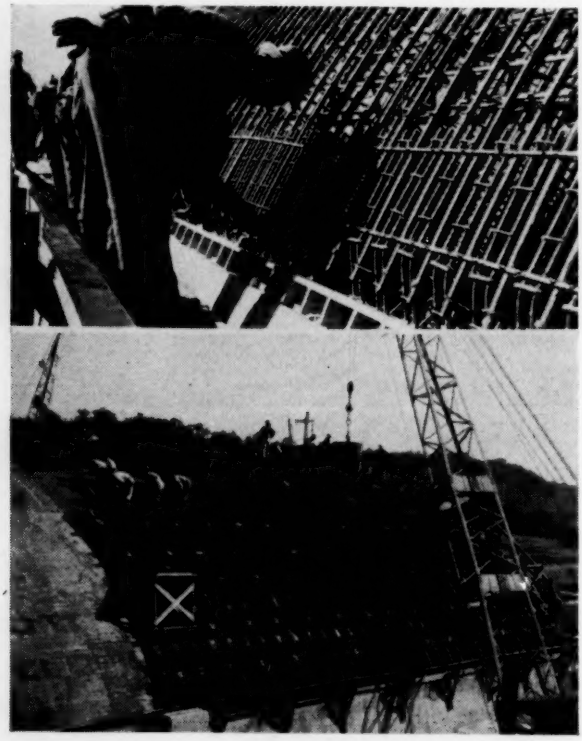

FIG. 13—STIRRUPS AND LATERAL STEEL BEING INSTALLED AT ABUTMENT HINGE END OF MAIN REINFORCING BARS

FIG. 14—GENERAL VIEW SHOWING BACKFORMS OF ARCH IN PLACE AND CONCRETE BEING PLACED

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centers thus automatically spacing the bars and being at a convenient height from the ground facilitated the placing and wiring of stirrups and transverse steel. Lattice trussing was welded into 5 lines of transverse steel by the contractor to provide the necessary rigidity for picking up the mats with a special strongback and transporting them by traveling crane to their location on the arch.

PLACING OF ARCH CONCRETE

Remarkably good progress was made in placing the arch roof of the basin building. After some delay on the first two sections the contractor placed the concrete for eighteen successive 60-ft. sections and the final 38-ft. section in nineteen weeks. High early strength cement was specified for the arch concrete and a compressive strength of not

FIG. 15—CROWN HINGE BARS INSTALLED, READY FOR POURING CONCRETE
OFFICIAL UNITED STATES NAVY PHOTOGRAPH

less than 3,000 p.s.i. was required before the centering could be dropped. This meant that a section of arch had to be placed during the latter part of each week in order to take advantage of the week-end for the setting of the concrete and that approximately three days of each week were available for dropping the centering, moving it 60 feet to the new position, plating it with the 2-inch thick insulation upon which the concrete was directly cast, placing the reinforcing steel and installing the backforms which were required for the lower 22 feet of the perimeter of the arch. This required thorough organization and substantial equipment.

The traveling centering was constructed of steel and mounted on trucks traveling on rails laid on the south wall of the deep basin and north wall of the high speed basin. When the traveling form had been moved forward in position for the next section it was jacked up to the proper elevation on fifty-five 30-ton jacks, one at each panel point of the traveler trusses, the weight of the arch as concrete was placed being carried on these jacks and not on the traveling trucks. In striking the form five men working from each side worked the jacks down gradually and evenly freeing the haunch-ends first. This operation took approximately one and a half hours, moving the traveler ahead, a half hour, and rejacking it carefully into position for the next section, about two hours. Placing of the insulation board started as soon as the traveler had moved clear of the previous section and was practically completed by the time it had been jacked to its true elevation.

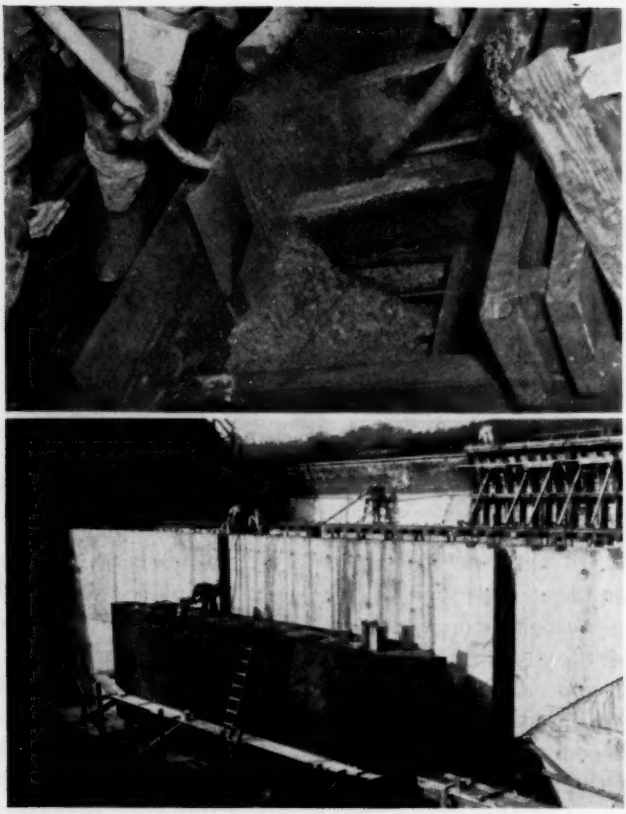

FIG. 16—VIBRATING CONCRETE THROUGH WINDOWS IN BACKFORMS
FIG. 17—EAST END OF DEEP BASIN SHOWING 2 FT. OPENINGS LEFT IN BASIN WALLS TO BE FILLED AFTER SHRINKAGE AND TEMPERATURE CONTRACTION OF 38-FT. SECTIONS (ALSO STEEL CAISON BEING ASSEMBLED IN THE BASIN)

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Two 20-ton whirley traveling cranes, one on each side of the arch then picked up the reinforcing steel which had been fabricated into mats as previously described and transferred the steel to its position on the arch. The crown hinge bar units were then installed in the holding jig which moved with the traveler. The stirrups at the haunch ends of the main reinforcing bars which could not be placed until these bars were entered between the hinge bar units, were then secured in place which completed the setting of the steel in about six hours after starting.

The backforms were heavily constructed in units 15 ft. wide and 12 and 10 ft. high. They were placed in position by the whirley crane

FIG. 18—INTERIOR OF BASIN BUILDING FROM STEPS IN WEST END OF TURNING BASIN BUILDING LEADING TO PHOTOGRAPHIC PLATFORM (OVERHEAD)

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on precast concrete spreaders and bolted through to the traveling form underneath. Four lines of removable batten openings were provided in the backforms for placing and vibrating the concrete. A 4½-to 5-in. slump, which was attainable with the maximum water cement ratio of 6½ gal. per bag, was used for the concrete within the back-formed area in order to eliminate any possibility of honeycomb. The backforms extended to the height where the rise in the arch was 6 in. per foot of horizontal projection. Above that point the concrete was dried to about a 4-in. slump and there was no difficulty in holding it without backforming although it was well vibrated. The whirley cranes were also used to place the concrete by means of 1-yard buckets bringing it up evenly on the two sides of the arch. By these means a 60-ft. section containing about 240 cu. yds. was placed in about five hours.

PLACING CONCRETE IN BASIN WALLS

The basin walls and floors were specified to be placed in sections 38-ft. long leaving 2-ft. wide openings between sections which were not to be filled in until after the building had been completely closed in and the basins were protected from large temperature variations. Fortunately this provision not only allowed from 2 to 8 months for shrinkage of the concrete in setting but also took care of the maximum temperature shrinkage to be expected since the filling of the openings was delayed until cold weather. The specifications provided that the

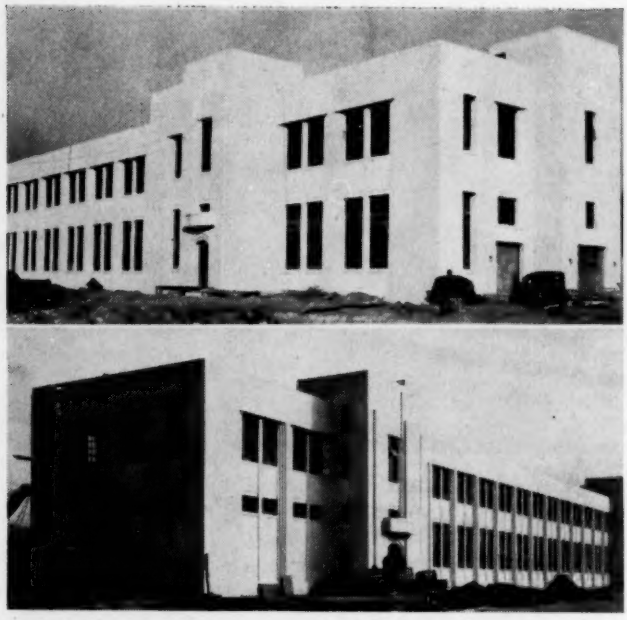

FIG. 19—LABORATORY BUILDING LOOKING N. W.
FIG. 20—SHOP BUILDING LOOKING N. E.
OFFICIAL UNITED STATES NAVY PHOTOGRAPHS

reinforced floor slabs which secure the upper part of the basin walls to each other and to the abutment walls of the arch should not be placed until after the basins have been filled with water for thirty days, thus eliminating future tendency to movement by reason of water pressure and on account of increased moisture content of the concrete. With these precautions it is expected that there will be no future movement to bring pressure on the abutment walls or require realignment of the carriage rail.

CONCLUSION

The foregoing brief description hardly does justice to a project which is so unique in purpose, which is the product of years of careful research and planning and which embodies so many unusual features of design and construction procedure. While the work is now nearly completed it is believed that an inspection of this project will amply repay engineers who are interested in concrete. A cordial invitation is extended to visit the plant at any time.

Discussion, to close in Supplement to be issued with Sept. 1939 JOURNAL, *should reach A. C. I. Secretary by June 1.*

Vol. 35 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

(Supplement)

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN *SEPTEMBER 1939*

Discussion of a paper by Hugo C. Fischer:

The Navy's New Ship Model Testing Plant*

BY J. C. PEARSON, BEN MOREELL, A. B. COHEN AND AUTHOR

CONVENTION DISCUSSION

J. C. Pearson†—This work is very interesting to the Institute's Committee 412, Architectural Concrete of the Exposed Aggregate Type. The building will be under the observation of this committee for a number of years, and some of us, at any rate, were concerned about this method of forming with pre-cast concrete which becomes essentially an integral part of the structural wall. We wonder, for instance, whether the cracks we usually expect in structures of great length like this, will or will not go through the facing or be transferred in some manner to the joints between the facing slabs? I should like to ask whether, up to date, this construction has been considered satisfactory and I would appreciate it very much if Admiral Moreell would answer the question and make any remarks he chooses in regard to the generally satisfactory trend of this type of forming and architectural treatment.

Ben Moreell‡—From the experience we have had with this particular project, that is, from the architectural standpoint, it has more than fulfilled all of our expectations. The appearance of the structure is extremely gratifying to those of us who worked on the design and who felt that in a structure of this size we were stepping out of routine, somewhat. The contractor has commented on the delay in his operations and undoubtedly this method of using pre-cast panels as forms

*JOURNAL, Amer. Concrete Inst., Apr. 1939; *Proceedings* Vol. 35, p. 317.
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for the structural concrete has resulted in delay. If the structure had been designed so that the pre-cast panels had been applied in the same manner as face stone, the contractor would undoubtedly have been able to make better progress and consequently a better cost record. We feel that, at least theoretically, there is an advantage in having monolithic character in the structure, that is, to have a facing secured monolithically to the structural concrete and acting as a part thereof. That advantage in non-earthquake regions is probably very largely theoretical. I am frank to say that if we were re-designing this structure or if we designed a similar structure which is higher, probably we would use the pre-cast panels as an applied finish rather than attempt to use them as forms; this for reasons of economy. The pre-cast panels would be backed up with tile, brick, or with concrete block. Now, with respect to the cracking, the last time I inspected the job there was one place where there was a very faint hair crack. There are probably other places, but I am inclined to believe that most of the cracking is concentrated in the joints between the panels. Commander Fischer will be able to comment on that point in more detail and more accurately than I can.

*A. B. Cohen**—I should like to know about the treatment of the key boxes in the basin walls for watertightness without using copper flashings.

Commander Fischer—A continuous vertical recessed key was cast in each end of the 38 ft. monolithic sections of the basin walls. Before casting the 2 ft. sections between the 38 ft. sections, there was applied to the interior corners of the key recesses a cove of bituminous mastic about the equivalent in sectional area of a ¾-in. chamfer strip. Reinforcement dowels also extended across the 2 ft. openings although the rest of the basin walls was not reinforced. The 2 ft. sections were not poured until the building was closed in some months after the 38 ft. sections had been completed and little further shrinkage was expected which would be counteracted by the moisture expansion when the basins were filled with water. Furthermore as the situation worked out, the concrete in the 2 ft. openings was placed at temperatures between 35° and 45° F. and since the basin building when in operation will be kept at a temperature of not less than 60° F. we always expect to have considerable pressure on these cold joints. (Note—In June after the basins had been filled for two months the leakage and evaporation combined averaged .001 in. per hour or about 4½ per cent of the total volume of the basins per annum.)

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A System of Concrete Control for Scattered Small Jobs, as Used by a Large Organization*

By R. B. Young and W. Schnarr†
MEMBERS AMERICAN CONCRETE INSTITUTE

SYNOPSIS

As far back as 1919 the Hydro-Electric Power Commission of Ontario has been applying quality control methods to its concrete work. While the knowledge of quality control could be applied to the big jobs, the scattered small jobs presented a different problem. But by 1926 jobs as small as a thousand cubic yards had been done using a modified control technique. From there on followed rapidly the development of "instructions" for superintendents and foremen on jobs not exceeding 500 cubic yards. These *Instructions* have now been revised in the light of more than a decade of experience and are presented by the authors—not as an answer to, but as a contribution toward the solution of the problem of quality control on small jobs. The Commission's operations are of considerable magnitude; it gives regular employment to a large staff; it maintains a large laboratory. Obviously the control methods which the Commission finds adequate and workable for its scattered small jobs would have to be altered for general application but these *Instructions* may, nevertheless, provide a key to the problem of "ordinary concrete."

The Hydro Electric Power Commission of Ontario was among the first to introduce modern methods for controlling the quality of concrete. As far back as 1919, a system based on the water-cement ratio theory was used in the construction of the High Falls Development‡; and by 1921, several jobs, totalling nearly three-quarters of a million cubic yards had been completed on which methods of control and inspection very similar to those practiced today had been used.

Naturally, on these early jobs, the technical problems were those of the pioneer; i. e., applying new principles to old problems and

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‡"Seven Years of Experience with Job Control of the Quality of Concrete,"—R. B. Young. *Proceedings* A. C. I. 1926, Vol. 22, pg. 79.

adapting new procedure to the variety of conditions that arose. During this period, the jobs were relatively large and no thought was given as to whether or not the knowledge and experience being obtained could be used on smaller work. However, as the success of the new practices became assured, this question arose and an attempt was made to solve it. By 1926, several concrete jobs as small as one or two thousand cubic yards had been built using in great part the control technique that had been developed for the larger operations. From this, followed a demand for modified methods that could be used on even the smallest concrete job and so instructions, applicable where the quantity of concrete to be placed did not exceed 500 cubic yards, were prepared for the use of engineers, superintendents and foremen responsible for this class of work.

These instructions were not immediately accepted by the men for whom they were prepared, and it has required not a little effort to convince them that they are reasonable and in the interest of the job; but that condition has gradually passed and now the methods have almost unanimous acceptance and are in use on all jobs for which they are applicable.

About two years ago it was decided to revise and amplify these instructions in the light of the experience gained in their use during the previous ten years. The resulting document, which forms the latter part of this paper, is offered as a contribution to the Institute's efforts toward bettering ordinary concrete.

These instructions are not submitted as an answer to the problem of controlling the quality of concrete on the small job. They are neither complete nor comprehensive and anyone reading them will find many things omitted that probably he would feel should be included but, in spite of their obvious omissions, they have worked well and filled a need in the work of the organization for which they were prepared, and that is the best answer the authors can make to possible criticisms.

It is obvious that these instructions have been written with a particular organization in mind. Because of this, it will be desirable to outline the conditions under which they have functioned so satisfactorily.

In the first place, the Commission does most of its own construction and this work is centralized in one department. It maintains also, a large laboratory, a section of which is equipped to make investigations required in selecting materials and testing concrete. The laboratory also carries out inspection of all kinds including that of concrete construction and so is staffed with men competent to deal with problems that arise in connection with concrete work.

The operations of the Commission are of some magnitude and its construction work offers regular employment to a large staff. The total number of men engaged in this class of work varies of course with its activities but there is always a sufficient number employed to insure a continuity in field operations, which facilitates the enforcement of instructions such as are here described. Periodically, there are newcomers to train but many of the foremen and others get their jobs by promotion and, therefore, are already familiar with the Commission's way of doing work.

To facilitate the training of the field forces and promote uniform methods, the Commission annually holds a foremen's convention to which are sent the more responsible field men who can be spared from jobs in progress. These men are from various lines of work; in fact, all of the trades that go to make up ordinary construction are represented. Some have daily contact with concrete work, some only occasionally, but because of the diversity of the work that they may be called upon to do and the probability that anyone of them may have to supervise jobs involving each of the different trades, all receive instruction in the concreting methods of the Commission.

With this introduction, let us consider briefly, the few simple requirements set out in the appended Instruction:

In the first place, it is intended both as a specification for the making of concrete and as an explanation of why it should be done as described. Both the specification and explanatory matter have been combined in one document because it is felt that the men will more willingly follow instructions if they understand the reason for them. To separate the "must" from the "why", all mandatory clauses are distinguished from explanatory clauses by the use of a distinctive type face.

The general specifications of the Commission require that any concrete exposed to the weather in whole or in part, shall have a minimum compressive strength at 28 days of 3000 p.s.i. For work coming within these instructions, this, in effect, is all concrete placed; thus only one class has to be considered, which helps to simplify the work.

For concrete where durability is the controlling factor, the practice of the Commission is to specify a definite minimum cement content. For average work having a coarse aggregate graded to not over a 2-in. maximum and subject to rigid inspection, this is unusually set at from five and one-half to six sacks per cubic yard. For small jobs, which receive no inspection and which usually have to use approximate methods for measuring the aggregates, the minimum

cement content is usually set at 6½ sacks per cubic yard.* This may seem high but it is more important on this class of work to make certain that the concrete is durable than to effect small economies in the use of cement. On an average small job, the cost of the extra cement thus used will seldom exceed $100, a sum which would not pay for many repairs, if they became necessary later by reason of poor concrete.

Throughout the instructions, special emphasis is placed on the necessity for keeping the total water in the concrete within the specified limits. To bring this out more pointedly, repeated reference is made to the water contained by the aggregate and the necessity of correcting therefor. This has been done deliberately because experience has shown a general tendency amongst concrete men to forget this important point.

Since the average small job has no facilities for testing the moisture content of aggregates, an empirical method must be used for its determination and so the instructions under Corrections for Moisture in Aggregates have been adopted. Experience over ten years has shown that this simple method is satisfactory and seldom presents any difficulty in operation.

Proportions are given both in weights and volumes but the latter are generally used in practice. It will be noted that no reference is made to the usual dry rodded volumes, the proportions being stated in terms of aggregates containing natural moisture, i. e., in the condition which they will be used on the job. This practice introduces no important error, yet it avoids the necessity for correcting the proportions for the bulking effects of moisture. In most cases, the aggregates are tested prior to use and the proportions are set by the laboratory but where, for any reason, this is not done, the proportions specified in the Instructions are used. As the Commission's practice on all concrete operations of any size is to measure all materials by weight, cognizance is taken of weight proportions for the benefit of those men who are used to working with these units and to permit the use of weighing where desired.

It will be noticed that the Instructions permit those in charge to modify the basic mix as conditions on the job may require. This is done to provide for variations in the grading of the aggregate and for any changes in workability that may be necessary in different sections of the work. The practice may be questioned by some as putting too much authority in the hands of the foremen or others, but since the specification contains a minimum cement content, a maximum water-

*The Canadian sack of cement weighs 87½ lb. gross.

cement ratio, a consistency requirement and the provision that "the amount of sand shall not be less than ⅔ of the volume of the coarse aggregate or more than equal to it", the quality of the concrete is safeguarded and no trouble has so far resulted.

The inclusion in the instructions of an actual example for the calculation of batch quantities needs little comment other than to point out that it has proven useful and also helps to emphasize that corrections must be made for the moisture contained in the aggregates.

The clauses covering materials are simple because the acceptance or rejection of both cement and aggregates rests with the Commission's laboratory. An exception to this is made in the case of purchases of small quantities of cement.

One of the most frequent and troublesome questions that arise is the use of cement that has become caked from storage. Some of the Commission's work is carried out at inaccessible locations and cement has to be transported and stored months ahead of actual concreting. In spite of every precaution, caking of cement occurs and much study has been given to the usefulness of cement in this condition. Tests have shown repeatedly that caked cement that will pass the simple test given in the Instructions will meet standard specifications and make good concrete.

Because of their importance, considerable attention is given to the question of aggregates. Pit run aggregates are forbidden although much of the work has to be done in areas where commercially produced aggregates are not available and the use of unscreened materials would be a great convenience. At one time this was permitted, subject to certain conditions, but even with rigid restrictions imposed, the practice was found to be unsatisfactory and for the last few years, has been discontinued.

No great difficulty has been experienced in getting suitable coarse aggregates, little reference has been made to them but there has been much confusion as to what constitutes a good concrete sand and so this information has been included, together with an excerpt from the Commission's Aggregate Specification, which is used where sand is to be purchased from local dealers.

The latter may interest some of our readers because it differs in important particulars from accepted standard specifications. It places a rather high minimum limit upon the amount passing the No. 48 sieve, specifying twelve instead of the usual five per cent; it permits five per cent passing the No. 100 sieve, and it definitely limits the amounts that may fall between the No. 8 and 28 or the No. 14 and 48 sieves. These requirements are based on experience with water and earth

retaining structures built with aggregates that would not meet this specification. Through the years, the concrete in these structures has shown a lack of durability that could be traced to porosity of the concrete brought about by the use of sands either lacking in fine material or containing too large a percentage of particles of approximately one size. Deficiencies of this type can only be compensated for in part by the use of additional cement and therefore for concrete that must be durable and watertight we are more and more insisting on the use of well-graded sands.

The sampling of aggregates is considered at some length because it is a frequent and very important duty which has to be performed by the field men.

Admixtures are treated very briefly. In general, they are forbidden but on occasion, with the permission of the engineer, use is made of calcium chloride as a hardening agent and when so used the amount to be added to the mix is definitely limited to two pounds per sack of cement.

Measurement is also treated briefly. Much experience with measurement by struck volumes has demonstrated that where conscientiously done, it gives satisfactory results. There has been no difficulty in getting this provision carried out, for the officials responsible for construction, because of their experience with weight proportioning on large jobs, realize the necessity for accurate measurements. The men on the job also, once they have used struck volumes, appreciate the advantages of having a uniform concrete to handle and place, and are willing to take the extra trouble necessary to obtain it.

Due to the fact that the mixing equipment furnished any job is entirely under the control of the Commission, no specifications are required for this part of the concreting operations except to point out that the batch must be turned over for a minute and one-half.

Curing follows standard practice but some comment is necessary. regarding the treatment of concrete in freezing weather. No chemicals are allowed to be used as an anti-freeze. This is already forbidden under Admixtures but is mentioned again because of the persistent propaganda maintained in some quarters, recommending calcium chloride and other salts as an anti-freeze. The idea underlying such practice is very attractive but it is a poor prop to lean on in even moderately cold weather.

The other clauses under the heading "Precautions in Freezing Weather," represent in essence the Commission's practice in cold weather concreting; namely, to keep the concrete mixes at reasonable temperatures and to give them protection as soon as possible after

being placed rather than to put them in hot. Finally, it is specified that "Suitable means shall be provided for maintaining all parts of the concrete at a temperature of at least 50 deg. F., for not less than 72 hours after placing;" i.e., the concrete just within the forms rather than the enclosure built without the forms is to be kept to the required temperature.

The section on surface finishes is somewhat longer in proportion than other parts of the Instructions. There are two reasons for this; —first, a great deal of difficulty has been experienced because of poor finishing, and second, it is desired that there shall be as much uniformity as possible in the finish of the Commission's concrete. No apology is offered for the methods adopted; there are other ways of obtaining satisfactory surfaces, but these have worked well and that is their justification.

The final section of the Instructions deals with the testing of concrete and follows standard practice. They are included because samples of concrete are regularly required from all but the very smallest of jobs and the sampling and molding of the specimens have to be done by the field men.

In conclusion, a word of tribute to the many men who are building the Commission's small concrete structures. In the ten years in which an attempt has been made to introduce adaptations of modern concrete methods onto the small job, the quality of the work has greatly improved. Whereas, 2000 p.s.i. concrete was the rule ten years ago, concrete made today will often run as high as 5000 p.s.i. with no increase in the quantity of cement used. The workmanship has improved to a point where honeycomb, fill planes and other evidences of carelessness are practically non-existant and the general excellence of the concrete is evident to even the most inexpert observer. All in all, the engineers of the Commission are well satisfied with the results that are being obtained.

(APPENDIX)

HYDRO ELECTRIC POWER COMMISSION OF ONTARIO

INSTRUCTIONS FOR CONCRETE ON JOBS OF LESS THAN 500 CUBIC YARDS

Quality

Unless otherwise specified, *all concrete shall have a minimum compressive strength of 3000 lb. per sq. in., at 28 days. It shall contain a minimum of 6½ sacks of cement per cubic yard and the maximum amount of water used shall not exceed 5 Imperial gallons for each sack of cement in the mix.* (Water-cement ratio by weight of 0.57).

Water-cement ratio

The strength and durability of concrete is adversely affected by the use of any water in excess of that actually needed to hydrate the cement. Therefore *the maxi-*

mum amount of water specified shall not be exceeded and if circumstances require a more plastic mix than the proportions as given permit, *cement as well as water shall be added to the batch* to maintain the quantity of water used per sack of cement (water-cement ratio).

The maximum water is the total amount entering the mix and *shall include the surface water carried by the aggregates, which must be deducted from the total in determining the water to be added to the batch.*

Corrections for moisture in aggregate

Whenever the moisture content is not determined accurately by test, *the surface water that is carried by fine and coarse aggregates shall be assumed to be as follows:*

Aggregate	Condition of Aggregate	Imperial Gallons per Cubic Foot
Fine aggregate	Damp	⅓
Fine aggregate	Moderately wet	½
Fine aggregate	Wet	¾
Coarse aggregate	Damp	No correction
Coarse aggregate	Wet	¼

NOTE: An Imperial gallon of water is here assumed to weigh ten pounds.

These quantities, multiplied by the cubic feet of sand and crushed stone or gravel in the batch will give the quantity of water *that must be deducted in determining the amount of water to be added to the mix.*

Proportions, general

Proportions may be given in terms of dry weights or damp volumes but for jobs coming within this specification, the latter will usually be used.

Proportions by volume

For average well-graded aggregates, containing their natural moisture, the proportions by volume will be approximately:

Cement	1 sack
Sand	2.20 cu. ft.
Coarse Aggregate	3.00 cu. ft.
Water Maximum	5 gallons.

Proportions by weight

For average well-graded aggregates free from moisture, the proportions by weight will be approximately:

*Cement	1
Sand	2.25
Coarse Aggregate	3.25
Water Maximum	5 gal. per sack of cement.

Where proportions are given by weight but measurement on the job is by volume, it will be necessary to convert the weights specified into equivalent volumes of the damp aggregates as used.

For average conditions and materials, *a unit weight of 90 lb. per cu. ft. for sand, (either moist or wet), and 95 lb. per cu. ft. for gravel or crushed stone (either dry or damp) shall be used in the calculation of batch quantities* unless actual tests have been made to determine the unit weights on the job or other weights have been specified by the Laboratories.

Example of calculation of batch quantities

A typical calculation showing how to convert weight proportions to batch volumes and also how to make corrections for moisture in the aggregates is given here as a guide.

*The gross weight of a sack of cement in Canada is 87½ lb. and is assumed to contain a cubic foot of cement.

Assume that 3000 lb. concrete is required and a two-bag batch is wanted. Let the proportions specified be 1 : 2¼ : 3¼ by weight and the aggregate available be sand and gravel, both damp.

The weight of the different materials for a two-sack mix will be therefore:

*Cement	2 sacks i.e. 2 x 87½	= 175 lb.
Sand	2¼ x 2 x 87½	= 394 lb.
Gravel	3¼ x 2 x 87½	= 570 lb.

*The gross weight of a sack of cement in Canada is 87½ lb. and is assumed to contain a cubic foot of cement.

Converting these to volumes, using the unit weights specified in the preceding section, gives:

Volume of sand, 394 ÷ 90 = 4.4 cu. ft.
Volume of gravel, 570 ÷ 95 = 6.0 cu. ft.

The maximum quantity of water is specified to be 5 gallons per sack; hence, for a two-sack batch, this will be:

2 x 5 = 10 gallons

From this must be deducted the amount of water contained in the sand. (If the gravel was wet the water in it would also have to be deducted. DAMP SAND CONTAINS ⅓ or 0.33 GAL. PER CU.FT. AND THE SAND IN THE BATCH WILL THEREFORE CONTAIN 4.4 x 0.33 = 1.45 GAL. As no correction has to be made for the small amount of moisture contained in damp gravel, the water to be added to the batch will therefore be:

10 − 1.45 = 8.55 or 8½ gallons.

The actual volumes to be used are therefore:

Cement	2 sacks
Sand	4.4 cu. ft.
Gravel	6.0 cu. ft.
Water	8.5 gallons

If the proportions are specified by volume, it is only necessary to multiply them by the number of sacks of cement to be used in the batch, correcting for moisture contained in the aggregates in the manner just described, to obtain the quantities to be measured.

Correcting the mix to meet job conditions

If, in use, the proportions given produce too harsh a concrete, the workability may be improved by increasing the quantity of sand and correspondingly decreasing the quantity of coarse aggregates, except that the amount of sand shall not be less than ⅔ of the volume of the coarse aggregate or more than equal to it.

If the mixes are too wet, the quantity of aggregate may be increased provided the mixer capacity will permit; if not, both the cement and water should be decreased by whatever amount is necessary to obtain the proper consistency. Vice versa, if the mixes are found to be too dry, the quantities of aggregates may be decreased or the cement and water both increased.

Consistency or workability

No hard and fast rule can be given for determining the consistency of the mix for it depends on the type of work and must be decided on the job. Generally speaking, *the mix shall be as dry as circumstances permit, yet it shall be plastic and workable so that it will place and finish well, but it shall never be so wet that it will segregate in handling.* Since any increase in the wetness of a mix, increases the amount of cement and therefore the cost necessary to meet the quality provisions of these specifications, *mixes that are wetter than absolutely necessary shall not be used.*

Cement

Shipments or purchases of cement in excess of 50 sacks shall be tested by the Laboratories and shall not be used until reported by them to be satisfactory.

Cement that has been caked in storage is still usable if, when pressed between the thumb and fingers, it readily powders. *Cement that does not meet this test shall not be used until its quality has been checked by the Laboratories and permission granted.*

Water

Water for concrete shall be clean and free from injurious amounts of oil, acid, aklali, organic matter or other deleterious substances. Potable water is satisfactory; other water shall be subject to approval by the Laboratories before use.

Aggregates, general

The engineers shall be advised of the source of aggregate to be used on each job. Preferably, an established source of aggregate shall be chosen, but where this is not possible or where the Commission has never used aggregate from the source in question, samples of all aggregates which it is desired to use, shall be taken and submitted to the Laboratories for approval.

Aggregates shall be separated into sand and coarse aggregate before being used. No pit or crusher run materials will be permitted.

Where screened aggregates are not available and screening has to be done on the job, a suitable screen for separating the fine and coarse aggregate would be a commercial ½" or its equivalent, mounted on a 2½ ft. x 4 ft. frame sloped to a batter of 8 in 12.

In selecting sand* it should be remembered that fine sand requires more cement to produce concrete of a specified quality, and very fine sand will not be permitted. A very coarse sand gives poor finishes and concrete that does not weather well.

**Purchases of sand shall conform to the following requirements:*

Grading

Fine aggregate shall be graded from fine to coarse within the following limits:

	Percent Passing
No. 4 sieve	95
No. 48 sieve	Not more than 30, & not less than 12
No. 100 sieve	Not more than 5
Weight removed by decantation	Not more than 3

Not more than 75 per cent by weight of the fine aggregate shall lie between sieves Nos. 8 and 28, nor between sieves Nos. 14 and 48.

Sampling aggregates

It is essential that samples be representative of the material taken, as it is on these samples that the suitability of the supply is judged.

If a deposit is worked as a bank or pit and has an open face, the sample shall be taken by channeling the open face. Care shall be taken to eliminate any over-burden or stripping at the top or any that has fallen along the face.

In sampling from belts, trucks or stockpiles, numerous samples shall be taken at various intervals or at various places about the pile.

A systematic collection of samples, should when assembled, contain a correct proportion of each size and be representative of the larger mass. When the accumulated samples have been taken, it will be necessary to reduce the total to approximately 75 lb.

This may be accomplished by quartering as follows:

Mix all samples together while still moist.

Pile in the shape of a cone.

Spread by drawing the cone from the centre outward with a shovel.

Divide the sample into four quarters.

Remove the two opposite quarters.
Remix the remainder.
Continue this procedure until the sample is the size required.

Each sample shall be shipped in a secure box or bag, and contain the following information:

Source of supply.
Quantity available.
Distance to nearest point where material is to be used.
Sampled by.
Where the material is to be used.

Admixtures

No materials other than portland cement, aggregate, water and reinforcement shall be used in any concrete unless authorized by the Engineer.

Calcium chloride, when permitted by the engineer, is to be used only for accelerating the hardening of the concrete and no reliance is to be placed on it as a protection against freezing. *It shall not be used in excess of two pounds per sack of cement in the batch.*

Measurement

Cement can be batched in whole or half sacks without further measurement. If other than a half sack of cement is required, it must be measured in calibrated boxes.

All fine and coarse aggregate shall be measured separately in suitable boxes provided with adequate handles so as to insure that the specified quantities are in each batch. Where larger quantities of concrete are placed, struck buggies and wheelbarrows may be used.

Water shall be measured in calibrated tanks or barrels by an approved method.

Mixing

Mixing shall be done in an approved batch mixer and the mixing time shall not be less than 1½ minutes after all ingredients are in the mixer.

Placing

The surface on which concrete is to be placed shall be clean, free from debris, laitance and surface water. The surface shall be saturated with water for some hours previous to placing the concrete. Where the concrete is placed on rock or other concrete, dry cement or cement paste shall be brushed into the surface for bonding. The first batch placed shall be mortar, obtained by omitting the coarse aggregate from the specified mix and reducing the water as required. Following the mortar, the concrete shall be deposited continuously and as rapidly as practicable from bottom to top, except where otherwise directed by the Engineer.

The concrete shall be deposited in the forms as nearly as practicable in its final position. The plastic surface should be approximately horizontal and any dropping, chuting or method of placing that segregates the concrete should not be used.

The concrete shall be compacted during and after depositing by tamping, vibration, or with suitable tools. The concrete shall be thoroughly worked around embedded fixtures, and into all parts of the forms. Light spading of the concrete next to the form will prevent honeycombing and make the surface smoother.

Compaction must be thorough, but the concrete should not be over-worked as this is detrimental.

Curing

All freshly placed concrete shall be protected from the weather. As soon as the concrete has hardened sufficiently to prevent damage thereby it shall be kept continuously wet by

sprinkling or otherwise for at least seven days. Moist sand is very satisfactory as a curing agent.

Precautions in freezing weather

All aggregates shall be thawed free from frost and ice before being used.

Salt or other materials shall not be mixed with the concrete for the purpose of preventing freezing.

Concrete placed at a temperature lower than 40 deg. F., shall have a temperature of not less than 60 deg. F., nor greater than 90 deg. F.

Suitable means shall be provided for maintaining all parts of the concrete at a temperature of at least 50 deg. F., for not less than 72 hours after placing.

Surface finishes

All surfaces to be finished shall be screeded as indicated on the plan. Following this a wooden float shall be used to depress all stones and the concrete allowed to harden until floating does not bring water to the surface. At this time the whole surface shall be given a final wood floating. Steel trowels shall not be used except where specified. Proper curing shall follow for seven days.

The face forms shall be removed as soon as practicable, all fins or other projections shall be carefully removed, and any voids or damaged places shall be immediately saturated with water, and filled with a mixture of the same composition as that used in the surface, and allowed to set.

The surfaces to be finished shall then be wetted and rubbed with carborundum brick, or other abrasive, without applying any cement or other coating, until even and smooth. When this has been done, the grout or mulch which has collected shall be thoroughly washed off. For surfaces not specified to be painted, the surface shall again be wetted and rubbed until a small accumulation of fine grained paste is produced. This paste shall not be removed, but shall be carefully spread with a moist whitewash brush, to form a uniform, very thin coating upon the surface of the concrete.

Concrete tests

Concrete test cylinders shall be made on all jobs of 50 cu. yds. or over, and on smaller jobs when specified.

One test shall be made for each 50 cu. yd. of concrete placed. Three cylinders shall be considered a test. Concrete for tests shall be collected from numerous batches, remixed only enough to produce uniformity and then moulded.

When making test specimens, each mould shall be one-third filled and then rodded 25 times with a ⅜" bullet pointed rod. This operation shall be repeated until the cylinder is filled. The top of the cylinder shall be screeded off and allowed to settle but no further finishing shall be done. Stone over 1½ inches in size shall not be put in the mould.

Test cylinders shall be stored in damp sand where the temperature shall not fall outside the range of 60 deg. F., to 75 deg. F. All test cylinders shall be sent into the Laboratories by express in time for the 7-day test.

The Laboratories will supply the field with moulds and test form reports as requested.

Discussion, to close in Supplement to be issued with Sept. 1939 JOURNAL, should reach A. C. I. Secretary by June 1.

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(Supplement)

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Discussion of a paper by R. B. Young and W. Schnarr:

A System of Concrete Control for Scattered Small Jobs, as Used by a Large Organization*

DISCUSSION BY F. N. MENEFEE, BENJAMIN WILK, E. S. LARNED, AND THE AUTHORS

CONVENTION DISCUSSION

F. N. Menefee†—Does the Commission allow the use of calcium chloride?

Mr. Schnarr—Yes, we use calcium chloride for acceleration, limiting it to two pounds per sack of cement.

In 1937 a small step was placed in front of a school and freezing precautions were taken by adding calcium chloride to the concrete. Last fall I photographed this step which had disintegrated one inch along the corner.

Benjamin Wilk‡—I wish to ask Mr. Young whether, at a temperature of 25° F., he would use calcium chloride as an anti-freeze?

Mr. Young—As you may judge from Mr. Schnarr's remarks, we think it is quite unsafe to permit any leeway unless you read the thermometer all the time.

*E. S. Larned***—Are the Canadian specifications for portland cement similar to the A. S. T. M. specifications in this country?

Mr. Young—Yes.

Mr. Larned—Have you found it necessary or advisable to modify that specification to secure the desired result?

*JOURNAL Amer. Concrete Institute, April 1939; *Proceedings* Vol. 35, p. 337
†University of Michigan, Ann Arbor.
‡Standard Building Products Co., Detroit
**Coplay Cement Manufacturing Co., Boston.

Mr. Young—We have not, because within the last few years we have built no jobs to which we think that modified cement would be applicable.

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN APRIL 1939

Concrete as an Architectural Material

The record of a joint meeting of the New York Chapter, American Institute of Architects and the American Concrete Institute—held as the final session of the 35th Annual Convention of the American Concrete Institute, New York, March 3, 1939.

In February 1937, at the American Concrete Institute's 33rd Annual Dinner, R. H. Shreve, of the New York architectural firm of Shreve, Lamb and Harmon, brought to A. C. I. a message from the American Institute of Architects. He said in part:

"As a director of the American Institute of Architects, I am authorized by its President to bring to you, the American Concrete Institute, the greetings of the architects and to extend to you assurance of the wish of our profession to work with yours for the realization of the great possibilities of your materials and methods of construction. As architects we should work with you much more than we do.

* * *

* * May I make a suggestion? Your sessions at this present convention seem to have been devoted very largely to the technical aspects of cement and concrete behavior; what would you think of planning for some future meeting a session at which you would discuss, perhaps jointly with interested architects, the subjects of well-designed masses, pleasing textures, desirable methods of revealing the aggregate, durability of surfaces and other phases of your work in which we might be of use to one another?"

In preparation for its 35th Annual Convention A. C. I. had the active cooperation of Arthur L. Harmon, President New York chapter, American Institute of Architects, and of other architects. This made possible as the final session of the A. C. I. convention in New York, March 1-3, a joint session, the evening of March 3, with the New York chapter, A. I. A.

Leopold Arnaud and Ely Jacques Kahn had prepared brief papers setting forth the architects' views and their concern for greater knowledge of concrete as an architectural medium. In response, four members of the American Concrete Institute each presented a phase of the subject.

In the absence, due to illness, of Mr. Harmon, Frederick G. Frost, Vice-president of the New York Chapter, A. I. A., and John J. Earley, President A. C. I., presided jointly at the session at which papers were presented as follows:

"CONCRETE FOR GENERAL ARCHITECTURAL USE"
BY ELY JACQUES KAHN
ARCHITECT, NEW YORK

"CONCRETE IN ARCHITECTURAL SERVICE"
BY LEOPOLD ARNAUD
DEAN, SCHOOL OF ARCHITECTURE, COLUMBIA UNIVERSITY

"REMARKS ON DURABILITY OF CONCRETE"
BY ROY W. CARLSON
ASSOCIATE PROFESSOR OF CIVIL ENGINEERING, MASSACHUSETTS INSTITUTE OF TECHNOLOGY

"COOPERATION NEEDED FOR ARCHITECTURAL CONCRETE"
BY A. J. BOASE
MANAGER STRUCTURAL BUREAU, PORTLAND CEMENT ASSOCIATION

"CAST STONE AS AN ARCHITECTURAL MATERIAL"
BY C. G. WALKER
ASSISTANT SECRETARY, CAST STONE INSTITUTE

"THE CHARACTERISTICS OF CONCRETE FOR ARCHITECTURAL USE"
BY JOHN J. EARLEY
ARCHITECTURAL SCULPTOR

The six papers form a group and are so presented, in the order in which their authors presented them at the joint meeting. They are recorded here by the A. C. I. as a step in the direction of that cooperation invited by the American Institute of Architects, through Mr. Shreve, two years ago.—EDITOR

Discussion of these papers should be submitted in triplicate to the A. C. I. Secretary by June 1, for group publication in the Supplement (to Vol. 35) to be issued with the Sept. 1939 JOURNAL.

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *APRIL 1939*

Concrete for General Architectural Use*

By Ely Jacques Kahn†

SYNOPSIS

Why do we not use concrete with assurance for exposed walls? The reason must be in our concern for appearance and the permanence of our designs. We, as architects in America, have been overwhelmingly conservative. Long after European designers were experimenting with new forms, materials and space arrangements, we held to classic traditions. We avoided healthy experiment, held back the broader use of materials which could long since have enriched our source of supply. The essence of resistance to broad use of concrete is first, in lack of knowledge of its possibilities; second, the architect's concern for permanence of effect; third, while conscious of the work of Mr. Earley, the architect does not wish to stop with surface ornament; fourth, the architect wants to know more of pre-cast slabs that fit into normal construction. There is still some mystery about concrete.

It is interesting to search ones own mind for reasons why concrete for general architectural use has not been ones first choice.

The architect assumes that concrete for foundations, concrete for floor arches and concrete for reinforced slabs is part and parcel of his routine. He does not remember, perhaps, that thirty, forty or fifty years ago, this was not so. We rediscover these facts when we are called upon to examine old buildings, here in New York, for example; those buildings that boasted huge brick foundation walls, cast iron columns and terra cotta floor arches. Concrete, we would assume, is here to stay.

Why, however, do we not use concrete with the same assurance for exposed walls? There is no question of strength, for we know the

*Presented at the 35th Annual Convention American Concrete Institute, New York, at a Joint Session with the New York Chapter, American Institute of Architects, Mar. 3, 1939.

†Architect, New York.

possibilities of reinforcement. The issue must lie, primarily, with appearance and our concern for the permanency of our designs.

To search a bit further into our mental states, we as architects in America have been overwhelmingly conservative. Long after European designers were experimenting with new forms, new materials and new arrangements of space, we held rigidly to classic traditions. That meant, simply, that irrespective of the use and program of the building, its appearance demanded, apparently, the use of architectural elements that would conform to a tradition of dignity such as we wished it. This resulted in factory buildings, for example, that presented purely industrial detail in plan and working areas with colonial brick and stone elements on facades that tried to cover the actual use of the structures; the result is that we have domestic colonial inspiration blown up to scale and use entirely foreign to the sentiment that produced the original buildings.

The same policy developed other modern structures, hotels, railroad stations, post offices and office buildings, to mention but a few that are jammed into patterns that might well represent good planning as to arrangement and service but in their architectural shell conform, as well as they possibly can, to various types of traditional forms. There are exceptions to this, as to other rules, but there is little question as to the fact that once you have let the variety of designer who adheres to this principle, free to work on a job that might conceivably demand a more practical and possibly a nobler conception, he has been prone to turn out architecture rather than good building.

One sees more of this conception in the design of bridges; where clean, sharp concrete form, integrally part of the bridge design, might have been discovered, we find lavish use of stone, cut, carved and hung on the metal frame as so much Christmas tree trimming. To carry this theory further, the architect has not infrequently found that his services were not desirable where contracting firms or engineering groups would quite logically take advantage of the situation and stress their sense of values, plus a desire for economy, and convince the owner that what he wanted was a building or a bridge and not a monument.

The result of all this somewhat strange development was a tendency to avoid healthy experiment on the part of those actually best fitted for the task. It has also held back the broader use of many materials that with the criticism and constructive analysis that experiments might have developed, could long since have enriched our sources of supply.

Some years ago, many of us employed cast stone on our buildings. The reasons were, simply, low cost plus endless possibilities of treatment as to aggregate, color and texture. We found in many cases that this concrete aggregate did not stand up. Cracks, discolorations and spawls warned us back to brick and stone. We knew, of course, that the major reasons for the blemishes were lack of seasoning of the blocks, too much rush in manufacture and, possibly, some carelessness. We doubted whether it was wise to risk further. Perhaps the manufacturers will advise us clearly as to the status of the cast stone now being made.

We have been concerned with absorption of dirt particles, cracking and general discoloration. Will the engineers tell us what to do beyond the usual precautions most of us know. We realize that we painted many of our earlier concrete buildings but paint is not the answer for permanency.

We are concerned with water absorption. What waterproofing preparations, if any, should we use that will avoid marring the surface and accomplish the results we wish? Manufacturers' statements are interesting but slightly prejudiced.

One particular use of concrete that has not been developed in our country as it should long have been, is the casting of textured forms that do not pretend to make concrete resemble anything else but concrete. It might be interesting here to comment on what Frank Lloyd Wright has done. In his houses on the west coast, he has produced the most delightful patterns, cast in simple molds and repeated so as to produce a delightful rhythm and wall surface. His houses are definitely of the material and whether or not everyone accepts all of his work as sacred, he rises like a giant above the platitudes of so many of his contemporaries. These wall forms present a design of moderate size as a pattern and develop a beautiful surface that seems to fit admirably into a garden setting. Wright has been so wise as to use concrete as a material and not as a subterfuge. It would be instructive to have designers more aware of the results that this brilliant designer has produced.

What, after all, is the essence of resistance to broad use of concrete? *First;* lack of knowledge as to its possibilities, bearing in mind the work of Wright as an example. *Second;* the architect is worried about permanency of effect. His restricted experience makes him judge concrete walls as an inferior substitute for stone or brick. *Third;* the architect, conscious of the decorative work that has been done by Earley, for instance, still does not want to rest on surface ornament.

He wants to know how to erect a permanent wall that has color, surface texture and design. *Fourth;* the architect wants to understand more about pre-cast slabs that fit into normal construction of every type. Perhaps all of these data are available, but in the minds of those who have been accustomed to the usual materials, there is still some mystery about concrete that those who sponsor it might elucidate. The problem of education of the young as well as the older architect is still before us.

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Concrete in Architectural Service*

By Leopold Arnaud†

SYNOPSIS

Unrestricted by the requirements of standardization concrete is adaptable as field-fabricated material or as shop-fabricated units. Its non-modular qualities make it eminently suited to contemporary building methods and design. It is an active agent in the changing form of architecture. There should be a moderation of the exaggerated requirements of our building laws to reduce cost and massiveness and increase the flexibility of design. The chief problem is with finishes—texture and color. What can be done to remedy the deficiency—of an extra operation for aesthetic effect. The needed improvements are: good integral texture, non-fading color, capable of accurate control and good weatherability.

Architecture, today, is in the early, vigorous, experimental phase of a renascence. Architects, today, must find it an inspiring vocation to carry their age-old profession through the turbulence of a vital regeneration.

The age of eclecticism is irrevocably past, and technological civilization has been recognized and accepted.

Nineteenth century eclecticism was an expression of assumed assurance, based upon an academic or archaeological familiarity with the expressions of the past, disguising actual bewilderment caused by the appearance of strange new forms consequent to the development of technology. But today, technology is seen as an extension of civilization rather than as a menace to it, and architects and engineers are busy, finding new applications and new aesthetic expressions for the discoveries that have revolutionized society. Truthful expression

*Presented at the 35th Annual Convention American Concrete Institute, New York, at a Joint Session with the New York Chapter, American Institute of Architects, Mar. 3, 1939.

†Dean, School of Architecture, Columbia University, New York City.

is consequently one of the fundamental tenets of the contemporary standards of aesthetics.

Technology has created new theories and new materials; they must be given practical application through suitable form.

Concrete is a material that has been known for centuries, but it has been so changed and improved by modern discoveries in chemistry, mechanics and processes of manufacture, that its potential uses have multiplied incalculably. Unrestricted by the requirements of standardization, adaptable as field-fabricated material or as shop-fabricated units, the non-modular qualities of concrete make it eminently suited to contemporary building methods and design. It lends itself to a truthful expression of its inherent qualities, and consequently is one of the most active agents in the changing form of architecture.

Despite the vast amount of building that has been constructed in concrete, the material itself and the *use* of the material are still in an experimental stage. But already a variety and high quality of aesthetic forms have evolved.

European designers have achieved a lightness and delicacy of effect in structural design which has not yet been ventured here, due in part to the requirements of our building laws. A moderation of the exaggerated margin of safety would lessen the cost as well as the massiveness of American buildings, and would increase the flexibility of design for concrete. It is to be earnestly hoped that these legislative changes will be effected.

There are other problems concerning concrete that have not been satisfactorily solved, either in Europe or in America, and which demand thorough scientific investigation.

The chief problem is the treatment of finishes—a difficult problem depending upon such variable factors as the design of the mix and the control of elements in field-fabrication.

Finishes may be considered under two aspects: texture and color. a) *Texture* has provided an active field for experiment, which has resulted in a variety of surface effects produced by a variety of methods: textures derived from the forms themselves or from lining the forms with stone, paper, oil, etc.; textures obtained by rubbing the surface after the forms have been removed, before the concrete has hardened completely; or by bush-hammering, sand-blasting, wire-brushing or cement-painting. Aesthetic effects of true beauty have been achieved by these methods.

Yet a disadvantage lies in the fact that the texture of the poured material rarely meets the requirements of sightliness except on very

large wall surfaces that are not seen at close range. The material therefore requires an additional operation which is not economical, and constitutes a serious defect in an otherwise admirable material.

What can be done to remedy this serious deficiency? Will it be possible to improve the method of pouring, or the mechanical manipulation of the surface, or the grain of the mix itself, to obtain a variety of surface textures that would not require further treatment? b) *Color* in building materials has always been an important factor in architectural design, for the architecture of every age has been poly chromatic. Architecture without color is nothing more than a recent and obsolescent misconception, and there is every indication that the architecture of tomorrow will be free in form, daring in proportion and bright in color. If concrete is to become a major building material, then color in concrete is a subject of major importance.

European architects have sometimes criticized American architects for the practice of obtaining rich or decorative effects by the use of fine materials. I believe that this is a biased criticism resulting from the fact that in Europe after the War, a great mass of building was erected by the cheapest means, with the result that modern work was conceived in terms of painted stucco and cheap surfacing. But contemporary form is not dependent upon these factors, and now as before, rich permanent surfaces are essential to fine architecture.

But the application of additional materials does not lessen the importance of the decorative quality of the main building material itself, for concrete will sometimes be used without incrustations, or else in combination with them.

At present the problems of color in concrete have not been mastered. One of the chief difficulties is *control in the field*, as *uniformity* must be obtained to produce good color distribution.

All concrete whitens or fades as curing progresses, due primarily to "covering up" by the release of the excess lime from the cement. Some authorities recommend the use of excess pigment so that the desired color value will remain after the whitening or fading has occurred. This procedure is too hap-hazard, and the results would be too uncertain. Moreover, the client would probably be highly dissatisfied if not skeptical during the first period of over-intensive polychromy.

The evidences are that concrete is potentially an excellent vehicle for color, and it is therefore legitimate to hope that the difficulties of adding pigment will be solved.

The "weatherability" of concrete is another important point involving considerations of expansion, contraction and shrinkage which must be included both in structural calculations and in aesthetic design.

The shrinkage that occurs during hardening and curing has less effect upon design than the provisions that must be made for expansion and contraction caused by changes in temperature—a matter of special importance in America where such a large part of the territory is subject to fluctuations of more than one hundred degrees. Assuming the theoretical coefficient of linear expansion of concrete as .000005, a wall one hundred feet long would expand approximately five-eighths of an inch, allowance for which would affect the design of the structure.

Contemporary design makes frequent use of decks and flat roofs. Good concrete roof slab is now obtainable, but due to this same problem of contraction and expansion, the slab must be covered by unsightly and expensive roofing materials. Expansion has also caused parapet walls to fracture and leak. For these reasons, concrete cannot be used to proper advantage in roofing, indicating that the material has not reached its potential development.

The "froth floatation" process is as yet too new to have been sufficiently applied and proven. What are the possibilities of improvement through this or other methods?

The American Concrete Institute is, of course, cognizant of all of these problems, especially the problem of shrinkage, for which reason the formation of a committee has been recommended to collect data pertaining to this important question.

This paper is offered, not as a contribution of new findings, but rather as a summary of the most needed improvements, stressing aesthetic considerations.

These are, in short:

Good integral texture, easily and economically obtainable;

Good color, non-fading and capable of accurate control;

Good weatherability, which will insure durability and resistance to drastic changes in temperature.

Improvements such as these would revolutionize structural design, and concrete would become one of the most important of all building materials, having an almost endless variety of potential uses.

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Remarks on Durability of Concrete*

By Roy W. Carlson†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

The author makes a conservative statement of factors affecting the durability of concrete: the fatigue action of volume changes produced by changes in temperature and moisture; the freezing of water in the pores of concrete; the phenomenon of bleeding; the nature of the constituents of concrete and their interrelation. Some concrete is durable under adverse conditions while other concrete is not durable. More concrete can be made durable—sufficiently durable for the demands of architecture when we come nearer the end of our search for a simple test for durability.

If all concrete were as durable as the best, and if we were able to predict durability reliably, we could be assured of ample durability for architectural concrete. The problem is mainly that of eliminating the element of chance from the selection of materials and the making of concrete. You have all seen concrete that has endured year after year with no signs of deterioration. In some cases, the concrete was not subjected to severe weathering, so no extraordinary quality was required. But in other cases, the concrete endured in spite of severe exposure. Such examples are proof that concrete can be made to withstand almost any exposure to which an architectural building material might be subjected.

The purpose of this paper is to discuss some of the underlying causes for the disintegration of concrete exposed to weather, and to suggest the need for simpler and truer accelerated tests for durability. The general subject of volume change is not discussed except in relation to durability.

*Presented at the 35th Annual Convention American Concrete Institute, New York, at a Joint Session with the New York Chapter, American Institute of Architects, Mar. 3, 1939.

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The destruction of concrete by weather is a type of fatigue action. Changes in temperature and moisture cause alternations of stress which the concrete must be able to withstand easily if it is to endure. Some parts of a concrete structure are being stressed and restressed repeatedly due to continued changes in temperature and moisture. The following explanation of the causes of these stresses is admittedly meager, but it may provide a suitable background for later discussions.

Firstly, temperature change causes stress, even when the change is uniform throughout a body of concrete. Whenever the temperature changes, the different constituents of the concrete tend to expand or contract by different amounts, causing highly localized stresses that may in turn cause invisible, internal cracks. In general, cement paste tends to expand and contract with temperature much more than does the aggregate. When the temperature change is not uniform through the concrete, additional stresses are caused by one part of the concrete tending to expand or contract more than another. The latter stresses may be more extensive and are likely to result in visible cracks.

Change in moisture content, especially loss of moisture, readily causes stress and cracking in concrete. The gel, which is the main cementing material that forms upon hydration of the cement, tends to contract greatly, while the other constituents tend to remain more or less constant in volume. If the graduation of particle sizes in the fresh concrete has been such that the gel has formed as a very thin skeleton surrounding inert grains of unhydrated cement and fine aggregate, the gel may not be damaged by the volume change. The mortar may then tend to contract as a unit. The contraction of the mortar is in turn restrained by the larger particles of aggregate and it is believed that the mortar is generally cracked between large particles of aggregate in concrete which has been allowed to dry in ordinary air. If the concrete is well proportioned, however, the internal cracks are likely to be extremely small, probably less than one-thousandth inch wide, and susceptible of being healed in time. A few remarks about this healing may be appropriate before proceeding to the consideration of other volume changes. The possible healing of fine cracks seems to justify the otherwise wasteful common practice of using fairly rich mixes and coarse cement, rather than lean mixes and extremely fine cement, so that there is always an abundance of unhydrated cement left in reserve. Unfortunately, the unhydrated cement is not readily available except as the cracks by chance open up the grains and expose the interiors to further hydration. The addition of a moderate amount of puzzolanic silica to concrete would seem to be better than to depend on unhydrated cement if healing of

internal cracks were to be a major consideration. The puzzolanic silica, and the lime necessary for its gelation, would always be available and the healing of cracks would seem to be facilitated.

A third major cause of volume change is the freezing of water in the pores of concrete. The expansion of the water upon freezing causes localized expansions somewhat analagous to the localized contractions of the gel upon drying. It is believed that water in the majority of pores in the gel of good concrete does not freeze, because the pores are of molecular dimensions. On the other hand, water in large, visible pores usually drains before freezing occurs and thus causes little damage. The dangerous pores seem to be those of intermediate size, probably invisible, such as are abundant in the gel of poor concrete and in many aggregates. In a recent series of freezing and thawing tests involving a number of variables, the most resistant concrete proved to be that which when mixed contained more than 10 per cent of air voids. The air voids were produced by adding less than $^1/_{10}$ of 1 per cent of a wetting agent to the cement before mixing. Attempts were made to saturate the specimens before freezing, and although it seemed that the concrete was more nearly saturated than concrete in service, there was some doubt about the saturation. This result, therefore, is only offered to show that visible voids are not necessarily detrimental.

No discussion of voids in concrete would be complete without reference to bleeding. Undoubtedly the control of bleeding offers a means of improving durability. The pores on the undersides of aggregate particles that result from bleeding are likely to be of all sizes, including those that are damaging. Furthermore, the lack of bond to the undersides of aggregate particles has other bad effects. This subject has been treated so well in the paper by Powers* that reference is made to that paper rather than to discuss the subject fully here.

When cement hydrates, the calcium silicate gel that forms substantially fills the spaces originally occupied by the mixing water. Attempts have been made again and again to make sizable specimens of such gel in the laboratory, for the purpose of measuring their properties. Invariably such specimens crack extensively soon after being exposed to ordinary air and they usually crumble upon a single drying. But hydrated cement is not all gel. The gel is interspersed largely with grains of unhydrated cement and crystals of calcium hydroxide, both of which are relatively volume constant. Thus,

*T. C. Powers, "A Study of Bleeding" also presented at 35th Annual Convention and scheduled for publication in the June A. C. I. JOURNAL.

as a result, larger specimens of neat cement can be made without cracking, because the gel has been diluted with the grains and crystals. The gel is still a problem, because, unless the water cement ratio is extremely low so as to keep the gel skeleton light, ordinary drying will cause plenty of cracking, even in specimens only a few inches thick.

In comparing neat cement with concrete, an interesting disclosure is that aggregate and its manipulation may be equally as important as the cement. When small neat cement specimens are prevented from drying and then tested for durability, they are found to be excellent. In fact, the neat cement, protected from drying, is better than corresponding concrete, in which the water cement ratio is naturally higher. In either case, whether neat cement or concrete, the durability is better the lower the water cement ratio. But when tests are made on specimens which have been dried in ordinary air for several months, the neat cement with its low water cement ratio is not durable. The neat cement disintegrates far more rapidly in a freezing and thawing test than does the concrete. And yet, very lean concretes do not withstand the durability test as well as do moderately rich concretes. The conclusion is simple but possibly important: Neither neat cement nor lean concrete is best, there is an intermediate mixture with some moderate cement content that is best. Just as the inert portions of hydrated cement dilute the gel beneficially, so does sound aggregate beneficially dilute the cement paste. Unfortunately, it is difficult to say exactly what proportions are best because there is still some doubt about what constitutes a dependable test for durability.

Considering the effect of maximum size of aggregate, a similar situation is found. Neither very large aggregate nor very small aggregate is best. Not only are the harmful effects of bleeding magnified where aggregates are large, but internal cracks due to volume changes are also worse. A lesser overall volume change is obtained with the use of large aggregates, but at the expense of greater internal stresses and cracking. Thus there appears to be some moderate maximum size of aggregate that is best, but it is difficult to say exactly what that size may be. In all probability, it will not be the same for all materials.

The effect of gradation of aggregate presents another similar situation. The general result of most methods of proportioning aggregates is that the combined aggregate is made as coarse as possible without exceeding the range of good workability. But this result can be obtained in a variety of ways. From the standpoint of durability, one gradation is probably better than any other. Present

indications are that the best gradation is such that the sieve analysis plots as a smooth curve, but even this does not fully define the gradation. The particular smooth-curve gradation which is best, remains to be found.

The physical and chemical nature of aggregate also have a great deal to do with durability of concrete. Some of the questions to be asked are:

1. Is the aggregate itself durable, or does it contain dangerous voids?
2. Does it have a high thermal expansion?
3. Does it change in volume appreciably due to wetting and drying?
4. What is its modulus of elasticity?
5. Does it necessitate the use of an excessive amount of mixing water in the concrete?

These are all questions bearing definitely on durability. A detailed discussion of them does not seem to be warranted here but it should be clear that there may be apparently-sound aggregates that will not make durable concrete.

Regarding the fineness of cement, there appears also to be a limit for best results. It was reasoned at one time that the main effect of finer grinding was to facilitate hydration. In other words, a lean mix with a very fine cement was considered to be about equal to a richer mix with a coarser cement. But recent studies of thin sections of concrete reveal a significant difference not acknowledged before. It has always been known that a large percentage of the grains of fine cement hydrate completely. In the case of the coarser cement, the numerous unhydrated cores of cement grains become the solid cores for the many little cells of unstable hydration products. The uninterrupted dimensions of the gel appear actually to be less in the case of the coarser cement because of the stabilizing cores. In concrete subject to drying, therefore, at least a part of the advantages of finer grinding may be offset by a greater degree of internal cracking.

The actual optimum fineness of cement must be dependent on composition, because only one compound (dicalcium silicate) can be relied upon not to hydrate completely under ordinary conditions. It is somewhat ironical that a good cement could not be made from this compound alone, even though it has wonderful long-time cementing ability and low heat of hydration. Besides having insufficient early strength, it produces nothing but gel upon hydrating and therefore it causes great drying shrinkage. Neither could an all-purpose cement be made from any other one compound in portland cement. Even the most desirable compounds, the calcium silicates, have an optimum proportion one to the other.

If the foregoing remarks are sound, it would seem that there is a good possibility that durability of concrete can be generally improved. If there is an optimum cement content, an optimum maximum size of aggregate and an optimum gradation, to say nothing about composition of cement and aggregate, our first task should be to determine these optimum quantities. There should be developed a simple and reliable procedure for testing concrete for durability. It does not seem that this procedure has yet been found.

Simplicity of testing procedure for durability is emphasized because the time and expense involved in some procedures prohibits sufficient testing. But simplicity must not be obtained at the expense of dependability. Tests on specimens which have not been subjected to the essential conditions found in service concrete are of doubtful value. In particular, at least a few tests should be made on specimens dried in ordinary air. If air drying causes internal cracks as numerous as one has reason to believe, then there must be doubt about the value of freezing and thawing tests on specimens which have been kept continually moist before being frozen.

Another question arises as to the saturation of concrete specimens before and during freezing. If service concrete is believed to be saturated before being frozen, then the specimens should be likewise. Soaking in water is not a sure method of saturating a specimen, because the entrapped air may be unable to escape. Saturation can be insured by placing under vacuum before adding water, or by heating moderately in water. But if service concrete is not likely to be saturated before freezing, the specimens might better be the same.

In view of the variety of conditions to which service concrete might be subjected, it is suggested that durability tests be made under several conditions. Not only should tests be made on specimens continually moist, but also on specimens dried for several weeks before freezing. Furthermore, tests should be made on saturated specimens as well as on those allowed to drain for a fixed time before freezing. Such tests are being tried in the laboratories at M. I. T. and early results look encouraging. Studies of volume change due to air drying are economically combined with studies of durability by using the same specimens for both purposes. Some specimens are frozen and thawed after their drying is practically complete and other specimens are frozen and thawed after having been moist cured continuously.

In conclusion, the exercise of caution is urged before making radical changes in concrete materials or proportions. It is almost a rule in concrete making that whatever change improves one quality is likely to hurt others. As a result, there have been a few backward steps in the struggle for durable concrete. But there has been progress.

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Cooperation Needed for Architectural Concrete*

By A. J. Boase†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

Thinking, on matters of reinforced concrete, is in a revolutionary state. There is much experimentation with new materials and techniques. Europe, with new techniques, is producing satisfactory architectural concrete. In these developments there is need of close cooperation of architect, engineer and constructor. An important consideration of structural behavior in building walls is in the serious restraints to volume change caused by foundations, floors, roofs, framework, and the greater stiffness at corners. The author discusses color, discoloration, erosion, textures.

As one who has been interested in architectural concrete for many years, long before he became associated with the cement industry, I am grateful both to Mr. Kahn and to Professor Arnaud for their many suggestions and expressed opinions.

At the present time, thinking on matters of reinforced concrete is in a revolutionary state. "Treated concrete" is being discussed freely with a view to field application. New reinforcing steels are being experimented with, as well as new design procedures. All of these things offer interesting possibilities for the future.

Other countries are erecting architectural concrete, using non-sliding movable forms and are finding them to be more economical than the standard methods being used in this country. Precast reinforced concrete slabs about two inches thick are being used for forming and are being incorporated into finished walls where color or textures are required different from those which can be obtained by the usual cast-in-place methods. In Europe they are using a different

*Presented at the 35th Annual Convention American Concrete Institute, New York, at a Joint Session with the New York Chapter, American Institute of Architects, Mar. 3, 1939.

†Manager Structural Bureau, Portland Cement Association, Chicago.

technique and are obtaining excellent results at costs which show real economy.

In all of these developments one point is clearly established—and that is the close cooperation of the architect, engineer, and constructor. Each plays a vital part in the building field, and real advanced will occur when each recognizes the other's possibilities rather than his limitations.

When the Paris architect Lubetkin conceived the continuous movable form, he recognized immediately the need for assistance from allied technicians and craftsmen. He called in his engineers and contractors, and between them they worked out an economical and successful solution. Each had to compromise and give ground, but the result was a building clearly expressing the friendly and honest cooperation of artist and technician.

In 1930, Sir Owen Williams started planning the Dorchester Hotel for London. He decided on precast slabs for the surface, backed up and bonded to the placed concrete. He immediately called in the architect and the contractor, and between them they produced a building which he showed me with great pride.

Harold Dyke, a London contractor, with the help of his architect and engineer, has devised a very efficient method of using precast slabs and it was found so economical that he has been given government contracts for the erection of several large housing projects in England.

In writing me recently, one well known architect who has done work in various parts of the country and is now on our west coast stated:

> One thing that has impressed me on the west coast is that we need the full cooperation of engineers of imagination to aid us in our efforts—and right from the take-off. For some reason, architects are inclined to feel that imagination is something beyond the mechanical realms of engineering, a path that architects of genius may tread unhampered by such annoying considerations as gravity, lateral forces and the like. Most of us are not engineers and cannot accomplish our "poems" alone. In other words, why should we hog the whole show? We can't play Uncle Tom, Little Eva, and the hounds as well. Architects from time to time have accomplished things that breathe the spirit of the cooperation of many minds working logically together —not the "priggish poem" of a single self-appointed genius.

No one person would attempt to answer all of the questions these two papers ask, and at the Chairman's request I am confining my remarks to a limited list.

STRUCTURAL BEHAVIOR

Both of the papers presented make preferred reference to the structural behavior of reinforced concrete walls. As buildings are

constructed today, the walls act principally as enclosures. The structural unit strength required for load bearing purposes is small.

The more serious stresses are caused by restraints to volume changes. These volume changes are due to the shrinkage of the concrete, expansion and contraction due to wetting and drying, and thermal changes. The restraints may be caused by the foundation, the floor and roof systems, the interior walls, the forms, and the increased stiffness at the corners of the walls where the return adds to the rigidity.

Probably the most important restraint is set up by the foundations where they are constructed in the usual manner. Here the concrete is usually buried in the ground, so that the moisture content is fairly uniform and the temperature changes are relatively less than in the concrete in the wall above grade. When the first story wall is placed and its initial shrinkage occurs, the basement walls and the forms act to prevent the necessary movement. When the forms are removed and the drying out process continues, the concrete is placed in tension and cracking occurs at the weakened section, usually at the corners of the openings.

Surveys of existing structures have shown that at least two-thirds of all of these defects occur in the first two stories and in the parapet wall where exposure is extremely severe. This would indicate that the floor systems, interior walls, corners and other restraints are of lesser importance than is the foundation wall. It would seem that the cracks in the lower stories of the building may act as articulations permitting movements to take place rather freely above.

In one particular job observed the basement wall was practically built of dry rock. The building is about 100 feet long, four stories high, and was built in 1906 in the northeastern area. That the building had moved on its foundations was evident, and no cracking could be observed in the wall although window openings are frequent.

Another job, built the same year and in the same location, was about 200 feet long, 10 stories high, and without openings or defects. The construction of the basement here was quite different. In this case there was no basement wall attached to the first story wall. Slender columns were used, and these probably took the movement by bending.

Walter Steilberg of Berkeley, California, by the use of models, has duplicated the conditions described above and his studies are illustrated. He made a model of a wall with a material having a shrinkage factor of 16 per cent (Fig. 1). The rollers, of brass, moved on a steel track. Over the brass rollers pieces of glass prevented restraint between wall and roller. Under this condition of free move-

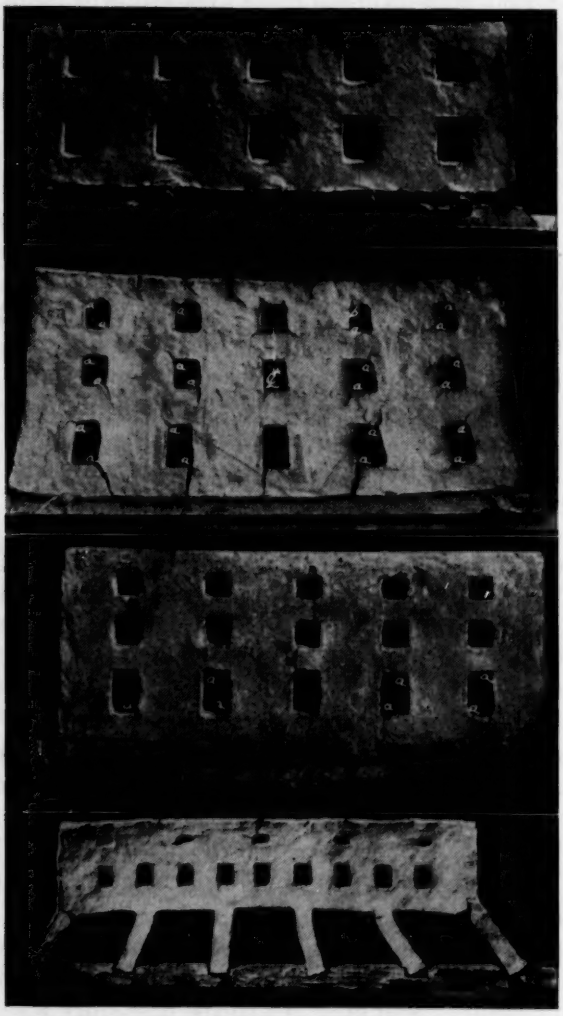

FIG. 1-4—TOP TO BOTTOM—SEE TEXT PAGE 367-8.

Walter Steilberg made a study of cracks in building walls. To make the effects more apparent he used, in a series of models, a material (bentonite) having a very high shrinkage factor (16 per cent) and observed the results in small scale walls in which various conditions of restraint were employed.

ment no cracks were formed. This is a similar condition to that of the building mentioned which was built on the dry rock wall, permitting easy movement of the building.

Mr. Steilberg made another model (Fig. 2) again with a shrinkage factor of 16 per cent. Here the wall was anchored to a resisting board by means of dowels on one-inch centers. The restraint set up by this board is similar to that caused by a foundation wall, producing a tendency, at the ends of the building, for the walls to curl from the foundations. Cracks formed at the corners of windows in a similar pattern to that of actual jobs.

A third model (Fig. 3) was of the same material as the first two except that the restraint is a 1:3 mortar kept moist while the wall was allowed to dry out. Here the restraint was even more positive than where the wall was doweled to a board, but the same general tendency prevailed. Such is the usual condition encountered in construction where the basement wall and the first story wall are attached.

A fourth model, made of approximately the same material as before, shows the condition of a building supported on slender columns which took the movement in bending. Result, no wall cracks. (Fig. 4)

These models with materials giving excessive volume change, are merely to demonstrate qualitatively how volume changes and various restrainst tend to deform a structure. No attempt was made to get a quantitative answer.

In an actual building at an institution where all of the buildings are of architectural concrete, dowel bars between basement and first story walls were exposed at the corners and over a period of time movements were measured. While no horizontal cracks occurred at the corners of the building, the measurements indicated definitely that with time there is a tendency for such buildings to curl at the corners.

Many attempts have been made to find a mathematical means of evaluating the stresses at any point in a wall subjected to known forces. No solution to date has been found, and it is not likely that one will be found in the near future because of the complex nature of the problem. Realizing the difficulty, a joint investigation with the Massachusetts Institute of Technology was started about a year ago. Professor Carlson, under whom the work is being carried on, has developed a laboratory technique, using rubber models and micrometer microscopes, which gives much promise. These experiments are being carried on as rapidly as possible and will continue until an answer is obtained or until it is definitely proved that the method is inadequate.

While these experiments may permit us to estimate these stresses quantitatively they will not eliminate them, and no matter what their magnitude, many things can and should be done to reduce them, such as more careful selection and grading of aggregates, the use of vibration which permits a dryer mix, slower and more careful methods of placing, more accurate field control, and better curing methods.

All of these things will assist materially, but in long walls articulation of a positive character should be incorporated in the design and construction. Structural engineers have for a long time recognized this principle and have successfully employed contraction or weakened plane joints as an effective crack control procedure.

The first architectural concrete job employing this procedure, as far as I know, was at Bellingham, Washington, and was developed by Floyd A. Naramore, architect, and A. M. Young, engineer. The practice has now spread on the west coast and has been further developed by T. C. Kistner, architect, and William T. Wright, structural engineer, and others. In writing on this subject, Mr. Wright states:

> This office has incorporated the joints in all of the concrete buildings designed in the past year and a half and it is believed that some advancement has been made in their use. From the writer's observations it would seem that weakened plane joints should be used (a) at openings, either side of wide solid sections of wall, (b) in all panels between pilasters thicker than the intervening panel walls, (c) in the exposed concrete surfaces where steel columns are cast into concrete walls, (d) in large wall areas at from 15 foot to 25 foot spacing, (e) and in large floor slab areas at intervals of from 15 feet to 25 feet, such joints being placed in line with the wall joints. The exact location in any case would depend on the actual design of the structure.

When the slabs are not exposed, it has not been found necessary to provide joints in the floor in line with the wall joints in order to make the latter effective.

A successful outside wall detail is shown in Fig. 5. The copper dam is punched for fastening to the reinforcing steel by means of wire. In this detail the mastic is not necessary to prevent leakage and may or may not be used, depending on appearance.

If this method is used, some engineers prefer to cut every other horizontal rod; others only weaken the concrete section, and results to date show that both methods are effective.

In all cases the weakened plane joints should be carried over the top of all window sills and parapet walls and caulked to avoid entrance of water.

Where floors and ceilings are to be left exposed, a detail such as shown in Fig. 6 should be constructed in the floor system. Where the floor is covered and the ceiling is hung, such a detail has not been necessary.

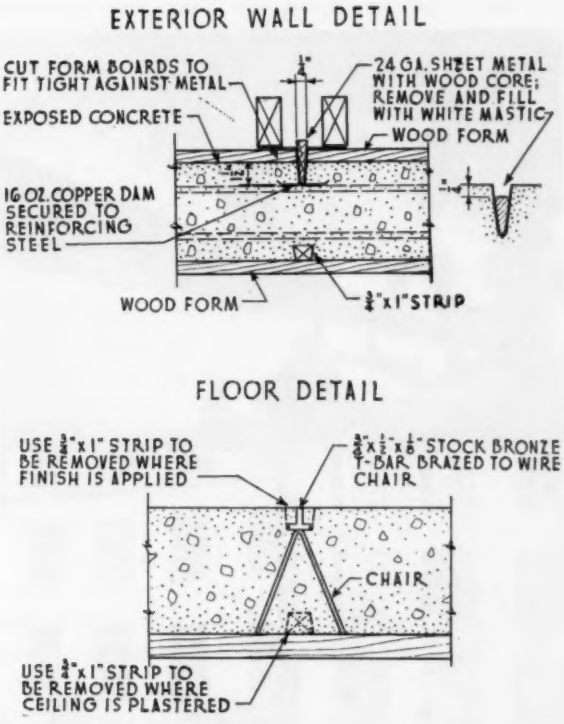
EXTERIOR WALL DETAIL
CUT FORM BOARDS TO FIT TIGHT AGAINST METAL
EXPOSED CONCRETE
24 GA. SHEET METAL WITH WOOD CORE; REMOVE AND FILL WITH WHITE MASTIC
WOOD FORM
16 OZ. COPPER DAM SECURED TO REINFORCING STEEL
WOOD FORM
¾"x1" STRIP
FLOOR DETAIL
USE ¾"x1" STRIP TO BE REMOVED WHERE FINISH IS APPLIED
¾"x½"x⅛" STOCK BRONZE T-BAR BRAZED TO WIRE CHAIR
CHAIR
USE ¾"x1" STRIP TO BE REMOVED WHERE CEILING IS PLASTERED

FIG. 5 AND 6

Each building should be studied as an individual problem, but the general rules given above should permit the intelligent location of these features.

Fig. 7 shows a wall of the RCA Building and the general appearance of these weakened plane sections.

COLOR

Professor Arnaud has raised the question of color being introduced in the concrete. One development in color now being studied may be of interest to the architects. It should be understood that the studies are by no means complete nor are we ready to say definitely that it will soon be ready for public release. Many details remain to be worked out and craftsmen trained for contractors desiring to employ the method.

Essentially the problem is one of casting integrally with the wall a surface of predetermined color, texture and pattern employing natural

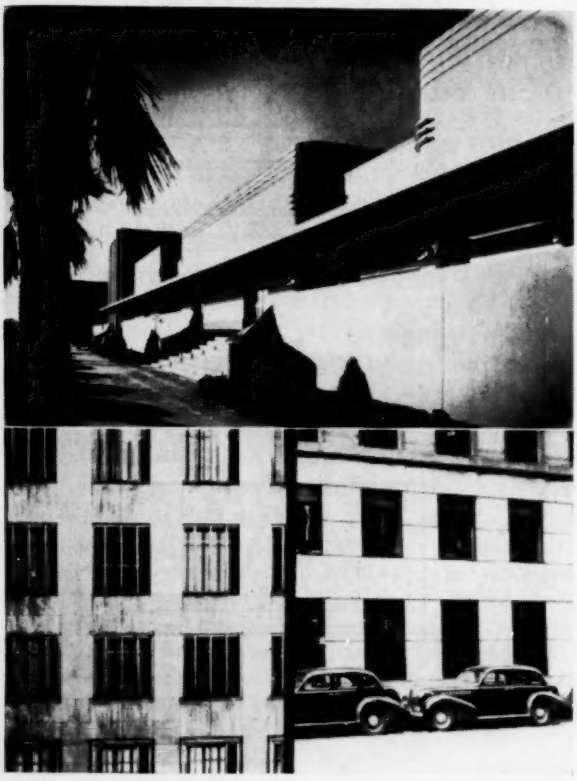

FIG. 7 (ABOVE) AND FIG. 8—SEE TEXT PAGES 371 AND 373

or artificial aggregates. For some time interested persons have felt that special aggregates could be applied first to the forms and then transferred to the concrete by means of differential bond.

The transfer method may be briefly described as follows: The facing material, either special aggregates or a thin layer of mortar containing the special aggregates, is uniformly distributed over form liner material, being bonded thereto by an adhesive. After a drying or curing period these prepared panels are fastened to the structural forms which have been erected in advance on the face to be treated The reinforcement and inside face forms are then erected and the forms are filled with concrete. After four to six days, depending on curing temperatures, the forms and then the liners are removed. During this curing period the aggregate or mortar facing has become securely

bonded to the concrete and if the proper technique is followed a 100 per cent transfer is obtained. After removing the liners the aggregate is exposed by rubbing the surface with abrasive stones, by sand-blasting or bush hammering. The final treatment consists of cleaning and brightening the surface with dilute acid followed by a thorough rinsing with clear water.

So far the best results have been obtained with aggregate passing the ½-in. screen shaped on the order of cubes. The experiments to date have been confined to aggregates of marble, ceramics and quartz.

Only simple designs have been tried and they have presented no special difficulties. The basic problems are of more concern at present and when these are satisfactorily solved more intricate patterns will be worked on until the method is perfected.

DISCOLORATION

The question of discoloration is brought out in both papers. I cannot believe that any light colored material will not discolor, and especially is that true in industrial centers where the air is laden with soot and industrial wastes from factories. This is an age-old problem of every building material. The real question is whether concrete discolors more than others.

Chicago is, I believe, as bad as most cities, and to get a comparison some investigations were made of concrete and comparable materials in exactly the same location.

Fig. 8 shows two buildings on the same street corner in Chicago just north of the loop. The one on the left is a well known building stone and is two years old. The one on the right is 14 years old, made of concrete.

The church building (shown in Fig. 9), erected in 1923, is in the heart of the Chicago loop district. The spire is made of concrete and the remainder of the building is a light colored stone.

The dome of the building in Fig. 10 is concrete, and the remainder stone. The building was erected in 1929. By comparison, under exactly the same conditions there appears to be little if any difference.

Streaking can be reduced if soot-catching projections are kept to a minimum. Drips as generally used will not solve the problem on any kind of building because of the action of wind. Even small ledges and sills can and have been sloped toward the wall and drained by concealed pipes. This will materially reduce erosion of flat surfaces as well as streaking. I assume that architecture is in this respect similar to engineering—about ten per cent of big ideas and 90 per cent of

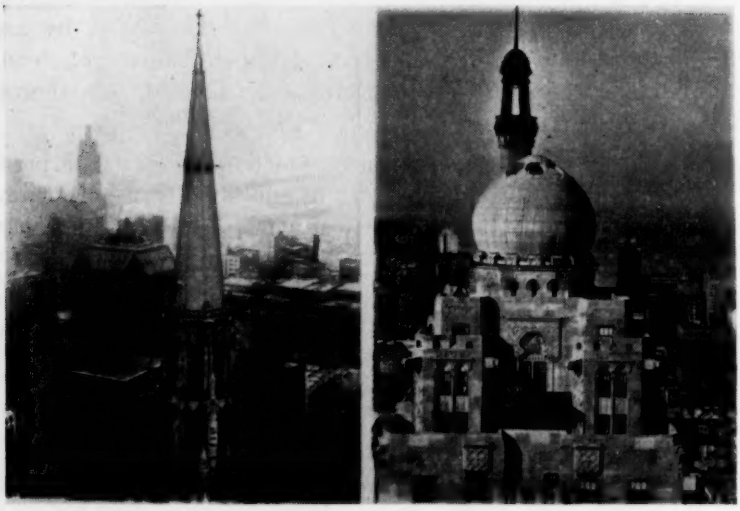

FIG. 9 AND 10—COMPARISONS IN GRIME COLLECTION
(SEE PAGE 373)

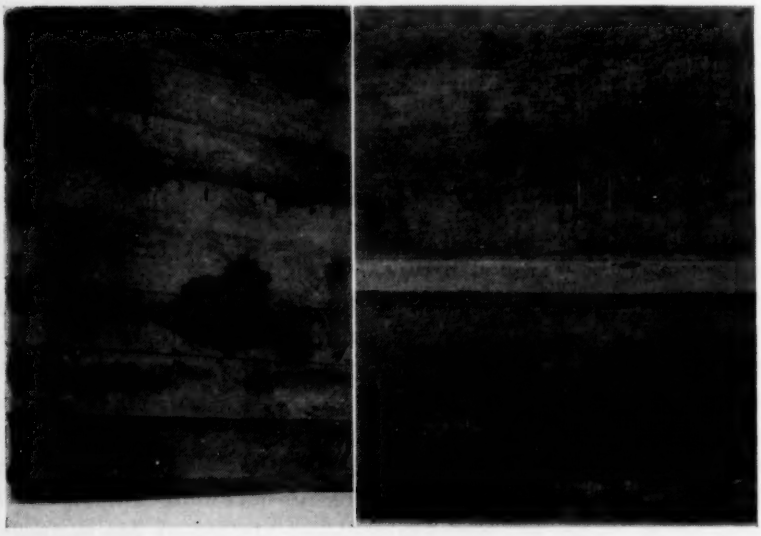

FIG. 11 AND 12—IN REFERENCE TO EROSION (SEE PAGE 375)

FIG. 13

detail. Thought will be required on these details for any type of construction.

EROSION

Both papers speak of preservation of form. Erosion of the vertical surface is not, or as far as I know, has not been a major problem. Where proper aggregates have been used and proper placing and curing procedure followed, even in industrial centers where the air is anything but pure, good concrete in a vertical position does not show erosion that can be detected by a visual examination. I have no doubt that any of us could point to buildings where the reverse is true.

Fig. 11 shows the west wall of a building at 800 Walnut Street, Philadelphia, erected about 40 years ago. The board marks are certainly distinct, and even a poor job of patching shows no effect of weathering.

That is not always true of horizontal surfaces such as ledges, copings and window sills. Here some slow erosion will take place, but I do not regard it as a major problem where reasonable care is taken to provide a dense concrete.

Fig. 12 is from Philadelphia, where the rainfall is 45 in. per year. The building is 12 years old. The ledge shows little if any surface trouble.

Another building in Philadelphia is shown in Fig. 13. This building is also 12 years old. Here the sills should show the greatest erosion because of the size of windows. Twelve years have not been enough to show any real trouble. I have no doubt that buildings could be found where erosion on horizontal surfaces presents a problem.

FIG. 14

TEXTURES

Both Mr. Kahn and Professor Arnaud have asked about textures. I am showing only a few because the number is almost unlimited and new ones are being tried out almost daily. Some are bad, some good—depending pretty much on the point of view and whether you are sympathetic or antagonistic.

Fig. 14 shows a fine arts center building in Colorado. Of this the architect wrote:

> The monolithic concrete exterior, revealing plainly the marks of the wide form boards used, was splendidly executed in keeping with the modern style of the structure. No painting was considered necessary since the natural concrete surfaces produce a warmth of color and texture in splendid contrast to the area surrounding it.

The surface shown in Fig. 15 was used by LeCoer of Paris on a school building for that city. Mr. LeCoer stated that he and his father had experimented with different materials for about 20 years and had finally settled on the architectural concrete style for their buildings. On this job the concrete was made from a rich red Belgian marble sand and a one-inch coarse aggregate, and after forms had been stripped and concrete hardened a sharp steel tool had been used to gouge the surface for a depth of about three-eighths inch.

Wide strips of plywood were used as a form liner on the building shown in Fig. 16. This surface is being used more than any other east of the Rocky Mountains. Of it the architect said:

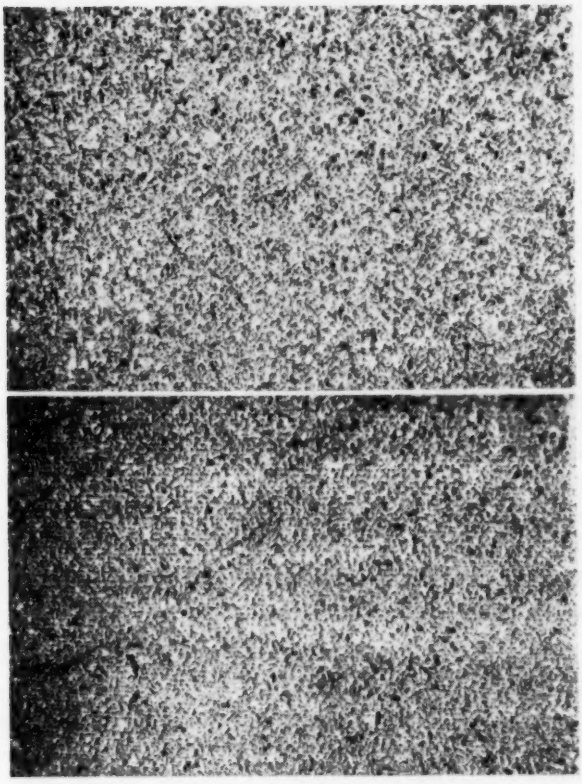

FIG. 15 (ABOVE) AND FIG. 16—SEE PAGE 376

Such imperfections as existed in the forms were not 'holy stoned' or fussed over. . . . Rather than attempt to imitate natural stone or to hide minor surface imperfections by plastering over them we deliberately permitted the material to express its true character of rugged strength and permanence.

A bush hammered surface was used for a boys' school building on Rue Kuss, Paris,designed by no less an artist than Mr. Expert. Mr. Expert stated that he preferred cast-in-place concrete bush hammered because it permitted him to vary the color by varying his aggregate.

Eight different textures were used to produce an ashlar effect—namely, Celotex, Rubberoid roofing, rubber ribbed mats, bush hammering, Sisalkraft, Presdwood (smooth and screen side) and wrinkled building paper. While the pattern is ashlar, the job on the ground does not look like an imitation of stone.

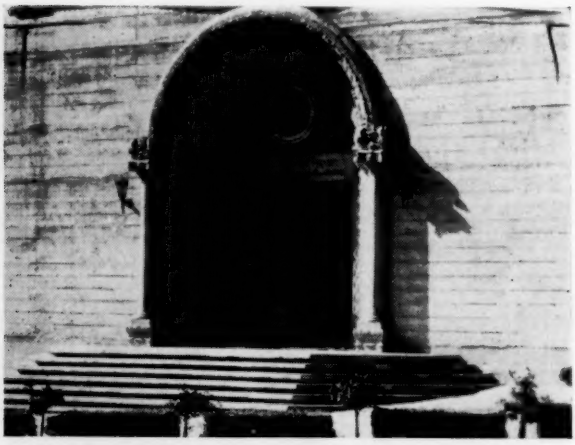

FIG. 17

In Germany the favorite textures are obtained by the use of stucco or by bush hammering. On a hotel building in Kassel the exterior is a limestone aggregate bush hammered. The ornament was cast in waste molds and later carved with a tool five-eighths inch wide.

In writing about the texture shown in Fig. 17 the architect who designed it stated:

> To obtain good scale and greater interest on exposed wall areas, we used ten-inch boards, taking particular care to soak the forms in order to raise the grain of the wood for pronounced texture. . . . There is a tendency for the wider form boards to cup slightly; but this very cupping improved the texture.

In conclusion, I can do no better than to quote from this same architect, a man who has used architectural concrete for a long time.

> It is the plasticity, the original utter formlessness of concrete that makes it so willing a material for the accomplishment of architectural purposes. It requires a bit of space and elbow room for placement, but beyond this minimum limitation it is free to assume whatever shape or form the designer chooses to give it. For effects of mass and strength it is incomparably well suited. Undoubtedly there are those to whom concrete, for architectural use, has a strange and alien look. True, it is comparatively new and in its mere beginning, and concrete is not yet familiar to all architects and all craftsmen, but it has life and novelty and presents the joy of fresh adventure. If some be cold to it, let Wordsworth's primrose be remembered: "A primrose at the river's brim, a yellow primrose was to him—and nothing more."

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *APRIL 1939*

Cast Stone as an Architectural Material*

By C. G. Walker†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

Cast stone started off on the wrong foot as an imitative substitute. Responsibility for its development must be accepted by those who sponsor it and inspiration must come from those who use it. Cast stone reflects the knowledge, experience and craftsmanship of its producer. There are as many degrees of proficiency and responsibility among makers of cast stone as in other industries—or professions. The need for discrimination in the selection of cast stone is no less and no greater than for other manufactured materials—and there are convenient means available for the needed discrimination in selecting cast stone. Ten years ago the general run of cast stone was testing under 3,000 p.s.i.; now manufacturers with any pretensions to reputation for quality accept 7,000 p.s.i. as a reasonable minimum for compressive strength. When there is difficulty in the application of concrete it might be best to adjust design and practice to the natural advantages and limitations of the material.

Mr. Kahn has mentioned various shortcomings of cast stone which have been revealed in its use as an architectural material. They undeniably have been real and serious. We know now that these difficulties sprang primarily from the lack of a sympathetic realization of the nature and limitations of the material. Unfortunately, in its beginning cast stone of the conventional type was started off on the wrong foot. It was given the role of an imitative substitute and it has not yet been released from that role. In this latter fact, to my way of thinking, lies the real misfortune and mistake. I am told that the first automobiles copied the form of the buggy even to the extent of having a place for a buggy whip and that early electric light fixtures looked like gas light fixtures. But automobiles and electric lighting

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†Assistant Secretary, Cast Stone Institute, Chicago.

soon escaped from the restraints of imitation and thus freed, the imagination of designers gave them distinctive form and character of their own. Cast stone, when released from its present bonds of imitativeness, offers no less possibilities. Responsibility for its development must be accepted by those of us who sponsor it but the inspiration and encouragement must come from those who are its prospective patrons.

Concrete admittedly is a durable material. Cast stone is simply concrete made in a factory. It should be concrete at its very best both in form and in quality because it is made under conditions which permit close control of materials and handling if the manufacturer is so minded and so required. When concrete in the form of cast stone falls short of what may be expected from this conception of it, the reason lies in laxity of the standards under which it was produced and used. I must frankly admit that on the part of many manufacturers there have been plenty of instances of such laxity as well as of carelessness at which Mr. Kahn so politely hinted.

The manufacture of cast stone is relatively simple but it is scientifically simple rather than mudpie simple. Being a manufactured material, cast stone inevitably reflects the knowledge, experience and craftsmanship of its producer, and in this country of rugged individualism there are as many degrees of proficiency and responsibility among cast stone manufacturers as in any other industry—or profession. Under such circumstances, the need for discrimination in the selection of cast stone is no less, and no greater, than in the case of most other manufactured materials.

Fortunately, there is a convenient and reliable means of exercising discrimination with respect to cast stone. Physical properties, as determined by simple tests, are a good index to the knowledge and thoroughness applied in the manufacturing process. The nature of cast stone work is such that all its producers are confined within rather narrow limits on mixes, aggregate gradings, etc. On such a basis, it is entirely logical to expect that the manufacturer whose careful observance of the fundamental rules of concrete making is reflected in high strength and density will be equally careful and precise in his drafting, pattern making, molding and all other phases of production. The requirement of high strength is, therefore, one means of making sure that cast stone will possess all the qualities which are to be expected in concrete at its best. With all due recognition and respect for the current feeling that the importance of compressive strength in concrete is being over-emphasized, we do not feel that a

better measure of the comparative excellence of different cast stones has yet been devised. There can be no question that under present-day open competition a practical and reliable measure of relative quality is indispensable to good results.

With respect to the status of present day cast stone, the range in quality has been extended by the full amount of improvements which have been made in it. In other words, certain rugged individualists are still exercising their privilege of manufacturing inferior material, and this will probably continue as long as they are able to find undiscriminating purchasers. On the other hand, it may be surprising as well as interesting to know that in routine tests on commercial cast stone we are finding compressive strengths up to 14,000 p.s.i. Ten years ago when the first standard specification for cast stone was being prepared by the American Concrete Institute the general run of cast stone was testing less than 3,000 p.s.i. Today manufacturers who have any pretensions to a reputation for high quality accept 7,000 p.s.i. as a reasonable minimum for compressive strength. Recognition of cast stone as concrete with the attendant freedom of expressing its own character opens the way to still higher standards.

I have referred to the handicap under which cast stone was originally placed by being required to look like something it wasn't. The handicap lay in the requirement thus imposed for using mixtures and practices which to say the least did not conform to the known principles of sound concrete technique. From the standpoint of appearance, concrete is characterized by particles of aggregate bound in a cement matrix. It is almost an axiom that the coarser the aggregate the better the concrete. Yet so far as cast stone has been concerned, the principal idea and effort has been to conceal its real identity by using aggregate so fine as to be practically invisible at arm's length. Overlooked was the fact that this course most generally produced a material subject to excessive shrinkage and crazing, high absorption and other defects which were mistakenly presumed to be inherent in cast stone.

It has already been suggested that honest recognition and treatment of cast stone as concrete is the first requisite for its successful use. On that basis, the importance of close observance of good concrete practice would be taken for granted. Beyond what this means with respect to size of aggregate and composition of mixes there is the question of the element of time. Perhaps the greatest problem with which we have to deal grows out of the failure of many producers as well as users to comprehend the importance of time as one factor in

cast stone making. Mr. Kahn mentions haste as being contributory to difficulties which have been encountered with cast stone in the past. No conscientious manufacturer ever hurried his product through the plant unless he was being hounded for delivery. It is possible in some degree and at some risk to compensate for lack of time by speeding up plant processes preceding and including the actual molding of the material. However, under ordinary conditions time is the very essence of curing and seasoning which are the next steps in the manufacturing process and it is the thoroughness of these which largely determines the performance of the finished product.

The idea that cast stone needs to be seasoned or dried out before being erected seems to be new in spite of its obviousness. At the end of the curing period, however long or short, cast stone and all other forms of concrete as well, contains excess moisture which is analogous to the sap in green lumber or that in freshly quarried stone. If cast stone is set in this moist condition the contraction inevitably attendant upon later drying may open up mortar joints and lead to discoloration or leakage to which objection has justly been raised by Mr. Kahn. Much ado over this natural phenomenon of shrinkage with loss of moisture is made in the case of cast stone and concrete generally. So far as I know, every solid body contracts when it loses moisture. That is why lumber is pre-shrunk by kiln drying before being used, why it is usually required that quarried stone be seasoned through extended storage above ground, and why the rules used in terra cotta plants have 12¾ inches to the foot. Since there is no mystery or secret about this natural phenomenon and no way of eliminating it, ordinary prudence suggests that provision against it be made by allowing cast stone to be properly seasoned or by using a pointing material which will compensate for shrinkage.

This is not the time or the place for an extended discussion of details in the manufacture of cast stone. As a matter of fact, from the standpoint of the user, details of manufacture need be of little concern. There has already been too much hullabaloo raised by competing manufacturers regarding the merits or weaknesses of different methods and sight has been too much lost of the fact that in the last analysis results are all that really count. I can assure you that the industry is amply served by plants competent to turn out cast stone meeting the high standards which have here been suggested for it is a highly refined form of concrete. Admittedly cast stone is neither foolproof nor perfect. I doubt whether it will ever be. When there is any difficulty in the application of concrete in its monolithic or precast form to architectural purposes perhaps it is due to efforts to make

it fit established forms and ideas rather than to adjust design and practice to the natural advantages and limitations of the material.

A walnut tree may be turned into a strictly utilitarian fence post or it may become an exquisitely hand-carved cabinet. Pig iron may go into a structural beam or it may be made into stainless steel. Portland cement may be turned into a foundation wall or it may be used to create a Baha'i Temple, a veritable Taj Mahal. Vision, skill and care are the transforming elements. Benefit of the contributions which concrete in the form of cast stone can make to architecture will be obtained only when it is permitted, even required, to be honestly expressive of its own character, when intelligent discrimination is exercised in its selection, and when its challenge to daring, initiative and imagination is accepted by architectural designers.

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *APRIL 1939*

The Characteristics of Concrete for Architectural Use*

By John J. Earley†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

Monolithic reinforced concrete, without a well designed and well executed surface treatment, does not even suggest the potentialities of color and texture which concrete affords. It is a mistake to think of concrete as a cheap material, or to impose on it the conventions frozen by other masonry materials. There is more difficulty in competing with the cheapest cut stone than with any better grade of material or workmanship. Concrete offers the best, economically, when the architectural requirements of form and color are most difficult. Such requirements take full advantage of the characteristics of reinforced concrete. The full development of concrete as an architectural medium is not solely in the hands of the concrete industry. The industry must add a sense of architectural beauty, scale, form, decoration and a sense of propriety. A reinforced concrete building may be successfully poured into forms made of thin, precast reinforced concrete slabs made by trained architectural craftsmen.

In California there has been a notable use of monolithic reinforced concrete, without special embellishment, in the construction of major buildings and there is evidence that its use will increase because conditions in California indicate, indeed they seem to demand, that type of construction. I received the impression in California that architects are convinced of the desirability and economy of monolithic reinforced concrete construction but realize thoroughly the unfinished appearance of these buildings. In mitigation of this statement, however, certain things must be considered and among them are these: The quality of light and shade found in California from San Francisco to the south is different from that found east of the Rocky mountains, excepting perhaps in Florida. It is more like the

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quality of high light and shadow we associate with the shores of the Mediterranean Sea. The high lights are brilliant, the shadows are deep and intermediate values are almost missing. Again, portland cement made in California is of much better color than that made in the East, excepting perhaps one plant in Pennsylvania. The western cements are more of a buff color, that is, a weak value of yellow-red as against the very disagreeable blue-green dyphthong of color found in the eastern made cements. Again, few masonry materials capable of receiving a good finish are found in California. The result therefore, is a marked tendency to follow the local architectural tradition which is to build with primitive simplicity and to ornament with individual pieces executed sometimes with great care and in fine materials, such as bronze, terra cotta, tapestry, and wrought iron from the great art centers of the world. These are often found decorating walls of adobe and rough plaster in a relationship that would be of questionable propriety in the east where finer finishes and materials are available.

It should be perfectly clear that monolithic reinforced concrete without a well designed and well executed surface treatment does not even suggest the potentialities of color and texture which concrete can afford to the designing architect and that monolithic reinforced concrete with the fullness of color and texture now available (please note that I refer to what is now available and not to what may be developed) is the most satisfying, facile and complete masonry material ever afforded the designing architect.

History informs us that every great impulse acting towards the modification of architectural design has been basically economic, and there are many reasons to believe that the present impulse acting to modify design is also economic. If this be so, a study of concrete for architectural use must be taken into consideration in relation to economy as well as technique. Economics do not seem to follow any definite movement; they act irregularly, and usually are related to some new idea; some apprehension of possibilities previously unknown. When the forms for monolithic reinforced concrete multi-storied buildings were first designed for many uses, an economy was effected which let concrete surge forward in an ever widening wave of usefulness which did not loose its impetus for fifteen years. It seems now that that wave has about leveled off to the same rate of progress as other materials and we are waiting for another wave.

In my opinion, it is a mistake to think of concrete as a cheap material, to expect to build with it any kind of building for any use for less cost than with another masonry. Long years of use have established the technique of other masonry materials and with the practical

exhaustion of their possibilities of design they have been frozen into conventions. To impose on reinforced concrete the conventions of other masonry materials is to rob it of its natural economic characteristics. Often architects and engineers are amazed at a cost for reinforced concrete, which is higher than the cost of other material and for a work which reasonably seems to be just the structure to be done with concrete. When such things befall our studio investigation usually shows that conventions have handicapped the concrete beyond what it can bear or that we ourselves, humiliating as it may be, have not yet developed our technique to the degree of perfection which displays the full possibilities of the material. Experience with the present degree of technical developments, teaches us that the economics of reinforced concrete and of other masonry are nearest to a balance when architectural standards are lowest. As architectural requirements are raised and esthetics become more of a major factor in the problem of design, the economy of reinforced concrete becomes more and more pronounced. There is more difficulty competing against wire sawn, run of quarry ashlar, than against any better grade of material or workmanship, and there's little difficulty competing, in fact there is no economic problem, whatever when the architectural requirements for form or color are extremely difficult. This is because such problems of design as involve extreme requirements of form or of color usually are solved by taking full advantage of the characteristics of reinforced concrete. Actual experience with the Baha'i Temple at Wilmette, Ill., and the United States Department of Justice Building at Washington, very well illustrates this point. The Baha'i Temple which represents an extreme requirement for form was estimated to cost $15,000,000. if executed in stone but it actually will cost $1,250,000 executed in reinforced concrete of the exposed aggregate type. Certain ceilings in the United States Department of Justice, which represent an extreme requirement in color and which comprised 15,000 square feet were executed with reinforced concrete of the exposed aggregate type for one fifth of the competitive cost of a reinforced concrete structure finished with conventional mosaics.

I see no difficulty in inferring that the full development of reinforced concrete as an architectural medium is not solely in the hands of the concrete industry, for the simple reason that the concrete industry does not seem to have a sufficiently sympathetic relation to architecture or to put it perhaps a little more bluntly, a sense of architectural beauty, scale, form, decoration and an inherent sense of propriety based on knowledge and experience with the worlds best architectural works. Such knowledge is not so generally distributed

through the concrete industry that we can say that the industry has that sense and it may be that the full development of reinforced concrete as an architectural material may be in the hands of the architects or in the hands of such architects as have an apprehension of a new style of architecture based upon a new material and who are willing to adventure if helped by the cooperation of suitable engineers and craftsmen. There are at present few architects willing to make such an adventure, and there are few engineers and craftsmen ready to cooperate and make the architects dream come true. Perhaps this is fortunate. It imposes a natural restraint against rashness and allows a new art to be developed thoroughly and reasonably successfully and not too painfully.

Some years ago I rebelled against the thesis developed in a committee study in the American Concrete Institute that the finish of a concrete structure, however fine it may be, must be integral with the structure. I maintained that a finish entirely distinct and separate from the structure was proper. We came to an impasse because, as I see it now, we were not able to produce a monolithic reinforced concrete building with high esthetic value and I could not decorate the building without a separate finish. The difficulty seemed to be like this: If a crew of men thoroughly trained as architectural craftsmen were employed to mix and place in carefully built forms concrete made from rigidly selected and often rare aggregate, the cost of a monolithic reinforced concrete building so constructed would be simply prohibitive. If on the other side, a crew of men organized for the greatest efficiency and economy were employed to place in ordinary forms concrete made with any structurally sound aggregate, it would be unreasonable to expect a monolithic reinforced concrete building of great beauty.

In the years that followed experience proved I was mistaken at a time when I had no solution to offer for the problem presented. I became convinced that the day of applied finishes had passed and set myself to find a solution for the problem.

It was customary to precast in the studio, all ornamentation incidental to the general decoration of great surfaces in the form of thin slabs. After the lapse of years we began to notice that all of these precast slabs were particularly free from cracks and other defects usually found on the surfaces of monolithic reinforced concrete buildings and of applied finishes. This attracted our attention and encouraged us to make a more careful study of thin precast concrete slabs. But the solution of the problem was not found until we conceived the

idea that a reinforced concrete building might be successfully poured into a set of forms made with thin precast reinforced concrete slabs. These slabs made with great care by thoroughly trained architectural craftsmen, in excellent forms with beautiful and sometimes rare aggregates, contain in themselves all the qualities desirable in a well designed building. They are the product of the designer and the craftsman. Their form is predetermined, so is the pattern of their surface, their texture and color. But in addition to that they are the forms for the structural concrete. It is poured into them, it is one with them, they are part of the designed thickness of the wall or the floor. They are not a veneer.

Precast slabs have already been used for an integral architectural treatment of reinforced concrete buildings of major importance. The present indication is that their use will increase rapidly and that they will be developed to a high degree of perfection. Already we have found that they can be made to control the cracking of reinforced concrete buildings, that they present a protective surface which is impervious and which retains for a long time its original appearance. While precast slabs present an immediate and satisfactory solution of the problem of decorating and protecting concrete surfaces, there is another potential solution which I hope will be developed in the not too distant future and which will afford an alternate means to practical though perhaps not equal results. It consists in the facing of forms with some kind of lining designed to impose on the surface of the concrete a designed, all-over-pattern, such as a diaper, which will effect a textural play of light and shade. The pattern should be pattern for pattern sake, there should be nothing in it reminiscent of the texture of other and well known materials and it can as architects know very well, be made expressive of scale and of a desirable variety. Furthermore, I am reasonably convinced that such surfaces can be colored with pigments without painting and without mixing pigments with the whole mass of concrete.

I do not know whether our studio will participate in this development because we are at present deeply engrossed in the perfection of thin precast slabs but the potentialities are there and will, I have little doubt, be developed. Craftsmen are already familiar with most of the necessary elements of a practical technique. All that is needed is a synthesis of the elements into a simple smooth technique. Its implication might quickly become general with the result that monolithic reinforced concrete surfaces, treated by a new technique for forming, or integrally incased in a precast exterior may become the most facile, complete and beautiful architectural medium.

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Durability of Pavement Concrete—Experience in Pennsylvania*

By Samuel W. Marshall†

SYNOPSIS

This paper reports data obtained from a comprehensive survey of concrete highway paving slab behavior in three adjacent Pennsylvania areas differing appreciably in climate. Quality of concrete aggregate also varies considerably in the different areas. Observations were made from standpoints of surface condition and structural condition of the paving slab. Three classes (or stages) of failure were recognized. In general check surveys confirmed original findings. A marked increase in rate of concrete deterioration is noted as climatic conditions become more severe. Concrete in all three areas is subjected to an appreciable amount of freezing and thawing but in one area the weather conditions are particularly severe. In the area of severest weather the quality of concrete aggregate is lower than in the others and heaving of subgrade is more prevalent. Existence of many factors influencing concrete paving slab durability is recognized but no attempt is made to interpret results of the survey in detail. Results suggest that present conceptions of the useful life of concrete pavement may require modification for localities in which many of the factors involved may be unfavorable to the lasting qualities of concrete.

The title assigned might lead us to believe that there is some standard or measure by which the durability of concrete roads could be determined. The literature on concrete roads indicates that some roads of this type have been in service 25 to 35 years. Probably there are others that would not be expected to serve as a concrete road surface for more than 75 per cent of that time.

The life of a concrete pavement may be expected to vary in accordance with the amount, type and volume of traffic, the severity of

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exposure to which such pavements are subjected, and the time and efficiency of construction. Variation in these factors precludes establishing a standard length of service for all concrete pavements. Recognizing this, the only way to evaluate the existing pavements is to survey the roads and classify the condition of the surfaces accordingly. This subject has been discussed miscellaneously, and all kinds of opinions have been reached, as to what the experience has been with this type of pavement. Such opinion, until based on actual conditional survey of areas representing a large mileage within given areas, will not give the true picture of the problem.

CONDITION SURVEYS

Several condition surveys of concrete pavements of Pennsylvania were carried on during the last year by engineers of associations interested from the standpoint of determining the economics of this type of surfacing. To use the results of such surveys as a basis for research studies, engineers from the Materials Division of the Highway Department of the State, made checks of the work performed by one association by covering the entire mileage of the concrete pavement in selected counties. A comparison of the two ratings was made, and where differences existed the classification given by engineers of the Department was used as a basis to establish a differential for the area involved. Two individual condition surveys were conducted—one covered 330 miles of roads constructed between 1924 and 1936 in eight counties; six in the northwest, one in the central, and one in the south central section of the State. The second covered 2960 miles of pavement constructed between 1916 and 1938 in thirty-one counties. The geographical location of the counties selected covered a wide band, extending in a diagonal direction across the state from the northwest to the southeast. As a check we selected counties which we felt represented the climatic and topographic conditions of the 2295 miles embodied in the survey of twenty-two counties. These 22 counties are divided into three groups and our check covered the entire mileage of one county in each group.

STRUCTURAL AND SURFACE CONDITION

On the original surveys the road surfaces were rated under three classes for their structural condition, and three classes for their surface condition. We felt that a structural condition is of value in determining the approximate life of a road, but the modern design of a concrete pavement has to a large extent corrected defects that might have been a factor in extensive cracking, and as much study is being given to structural design of concrete roads it was felt that this phase of dura-

bility was well recognized. Our check work covered and classified both structural and surface conditions, but surface defects as indicated by scale and disintegration were considered of major importance and of special value as a basis for this paper.

SURFACE CLASSIFICATION

Arriving at classifications is a difficult problem, in that there is a tendency to establish many classes within narrow confines which would require long time detail studies rather than reconnaissance which was considered desirable. The three classifications selected are rather broad, in substance as follows:

Class one road—a road practically free of scale other than thin mortar finishing scale, structurally sound, which should give a service as a concrete road for probably 15 to 25 years.

Class two road—a road having some scale which might be extensive, but which is not considered to effect the road structurally. This road should have a life as a concrete road of probably 10 to 15 years.

Class three road—a road having considerable scale and some disintegration which has had a major effect structurally. This road should be resurfaced immediately or within 3 years at a maximum.

A few pictures will show more clearly than description the general condition of the roads in the different classes. It must be appreciated in evaluating the road surface condition in any one class that such condition on any road under traffic is not uniform.

Fig. 1 shows class 1 road which is in a sound condition; if scale exists on such surface it is confined to the thin finishing variety. A road in this class requires only normal maintenance of joints. The photograph represents a road somewhat better than the average of the class. This road was built in 1923-24 under a contract covering 6.59 miles. This photograph was taken in Dec. 1938 and as may be seen, the pavement is still in excellent condition.

The Class 2 road surface (Fig. 2) has some scale, and at places this may be extensive, but if it is of a progressive type the progress is slow. Such a road requires much more maintenance, especially at the joints, than the Class 1 road. This picture represents what would be the worst condition on a Class 2 toad. The road was built in 1926 under a contract covering 4 miles. The pavement shows extensive surface and progressive scaling on hills. The picture was made Oct. 18, 1938.

In Fig. 3, showing a Class 3 road, there is evidently much more scale of the progressive type. Such scale has usually progressed to a stage where it affects the road structurally. Roads in this class usually serve as a base for resurfacing, but they may require some small amount of replacement with concrete. This road was built in 1926

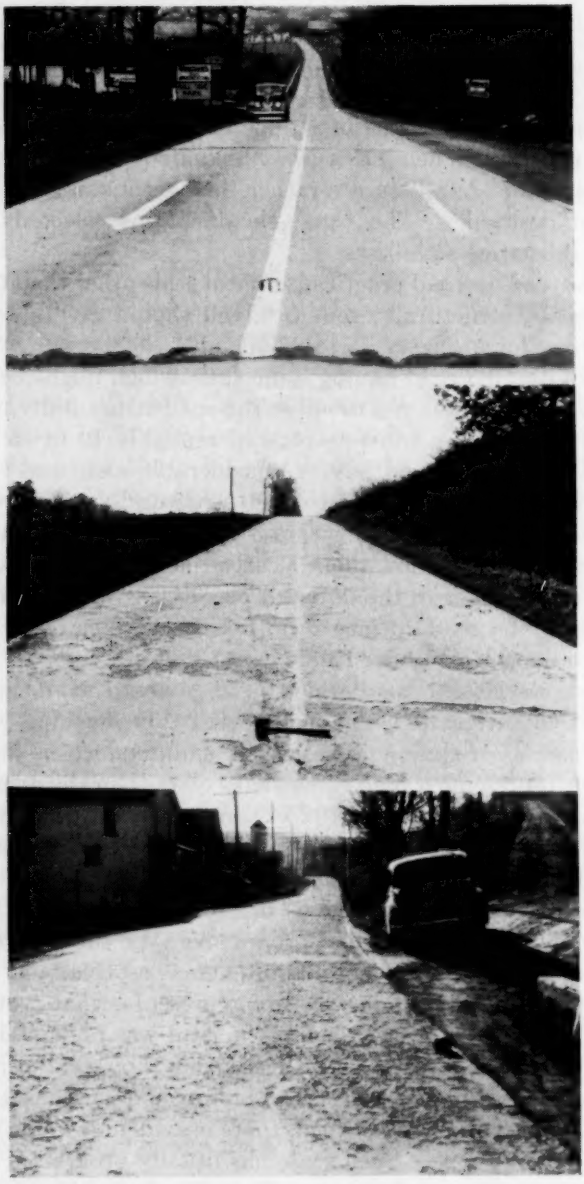

FIG. 1, 2, 3 (TOP TO BOTTOM) SEE TEXT PAGE 395

under a contract for 2.3 miles. The pavement is in very bad surface condition. The picture was made Nov. 11, 1938.

Resurfaced roads—One problem involved in classification was whether the roads which had been resurfaced should be considered. It was finally determined that since the concrete surfaces could not be observed, they should not be considered. The decision was not made until it was determined that a large number of the earlier constructed roads were not resurfaced, and the original surface could be observed. It was also considered that there was some probability that when a road was resurfaced it was not entirely due to the lack of durability of the road surface, as some resurfacing had been done to improve the riding, and it was known also that limited defective areas sometimes made it necessary from the general construction standpoint, to resurface contiguous sections of pavement that were not defective. In the areas involved there had been originally 2292 miles of concrete roads built between 1916 and 1938. At the time of the survey 403 miles of this had been resurfaced with a bituminous surface, the old concrete road functioning as a base.

RESULTS OF THE SURVEY

A relief map of the State, Fig. 4, shows that there is a considerable variation in topography. Some of the roads on high elevations are subject to severe exposure and we found in some locations the temperature changes were very rapid. In this latter regard on the Allegheny plateau in the northwestern area, special temperature records were taken over a period of five years. These plottings will be discussed later.

In Fig. 5, a State map, we have plotted the areas included in the condition survey, namely, A, B and C areas. In each area it will be noted there is one county designated as a key county. This is the county where our engineers made a survey of all the roads as stated before, in order to check conditions as recorded in the area.

Below the map is a graphic presentation of the per cent of roads in each class in the three different areas. The first three columns represent the structural and the next three the surface conditions as recorded. It will be noted that in both cases the percentage of good roads increases as we go from area A to area C. The surface condition columns especially show the preponderance of class three roads in area A, in the northwestern part of the state. The columns labeled "resurfacing" show the percentage of resurfaced concrete roads. Such resurfacing generally consists of bituminous material, and the old concrete road acting as a base. The position in the column has no reference to the class to which these roads belonged prior to resurfacing.

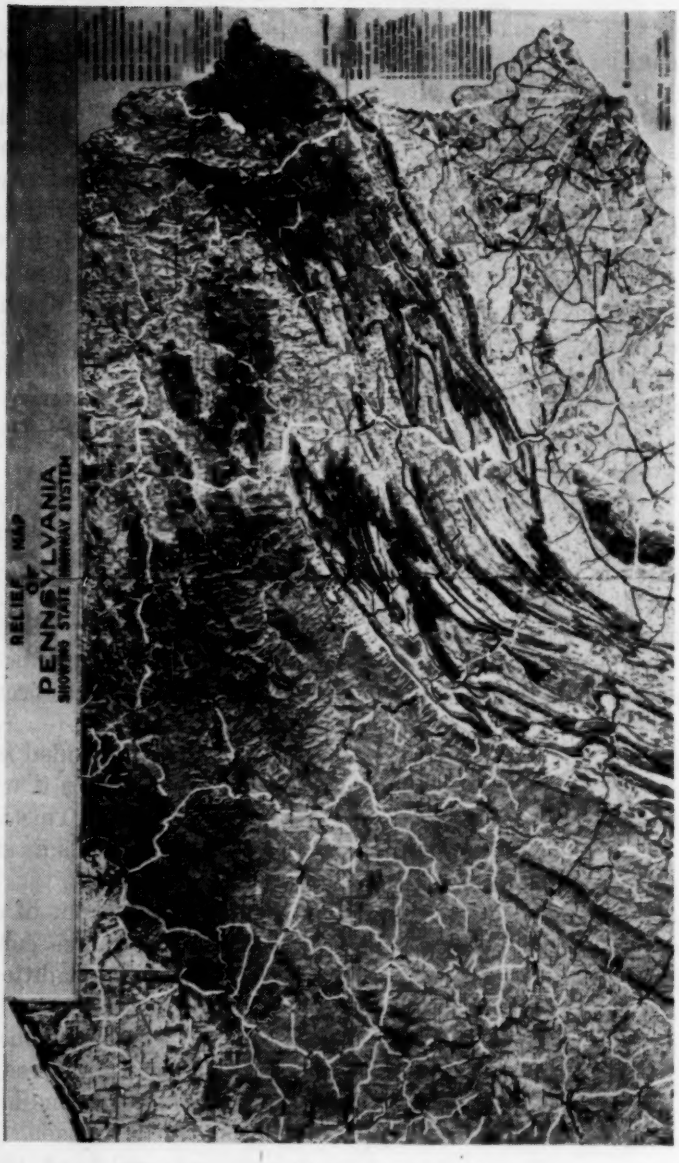

FIG. 4—THE HIGHWAYS SHOWN IN WHITE ARE CONCRETE

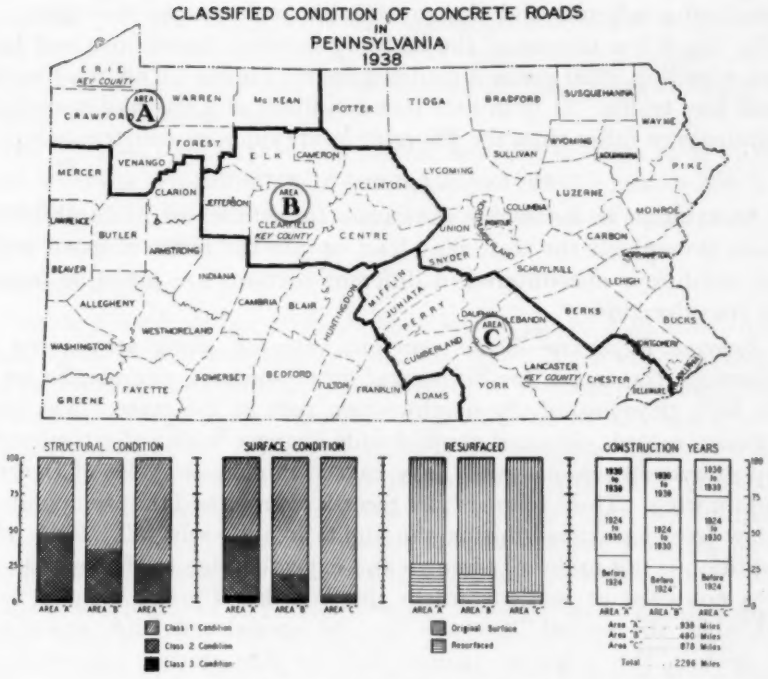

FIG. 5—SEE TEXT, PAGE 397

FIG. 6—CONCRETE ROAD BUILT 1920 FOR MAIN LINE TRAFFIC IN GOOD CONDITION WITH LITTLE MAINTENANCE

Attention is called to the fact that there is no relation between the resurfacing columns and the construction columns to the right.

In Fig. 6 is a picture of the highway between Harrisburg and Lancaster built in 1920 under a contract for 6.81 miles. This road carries main line traffic. It is in excellent condition and has had very little maintenance other than the filling of joints since its construction.

PROBABLE CAUSES OF FAILURE

An attempt to determine the causes for differences in durability in roads throughout the state may lead us into the realm of guess work, but conditions encountered in different sections are probable factors for consideration.

Extreme Exposure—Some previous research work attempting to determine the causes for disintegration of concrete was carried on in the high elevation of the northwestern part of the state. The temperature records obtained showed wide changes within short intervals and at certain seasons must have caused numerous cycles of freezing and thawing. These temperature records also showed that temperature below freezing existed during the nights in the early fall. Since this condition is not ordinarily known and expected, due to the fact that it does not exist in sections where the Weather Bureau records were available, the probabilities are that no special provision was made to protect the concrete against this freezing during construction. Freezing would hardly be expected when the day temperature was 80° or 90°. The effect of this surface freezing of the pavement may not indicate itself at the early ages, but might be a major cause of surface weakness leading to scale.

Subgrades

The roads in the northwestern area are subject to deformation due to heaving of subgrades. This has been recorded where levels were run, and different elevations of slabs observed at different times of the year. We believe that such deformation of the road may be a factor in producing defects.

Aggregates

The fine aggregates in the northwestern area, although passing all test requirements, generally are not as structurally sound as the aggregates used in other sections of the state, and in some cases this is true of the local gravels. It has been found necessary to have closer control on soundness to limit the use of shale in fine aggregates, and to run frequent Los Angeles abrasion tests to control the gravel. The aggregates cannot be considered solely as a direct cause for disintegration, due to the fact that a road built in the central part of the state

with fine aggregate taken from this northern area has shown excellent durability. If the use of such fine aggregate is a factor its effect is involved with other conditions, such as aggregate and cement combinations, stresses produced by subgrade heaving, frequent cycles of freezing and thawing, etc.

The cements used in this area have a wide use in other parts of the state where generally the roads are good. Therefore, if cement is a factor, it must be associated with the combination of aggregates or other special conditions.

Structural Defects

Core drill explorations at joints have shown porosity on some roads constructed with dowels used for load transfer. No doubt weakening, due to porosity has resulted in the breaking of the concrete contiguous to the joint. Due to the fact that disintegration usually starts at a joint, this might be attributed entirely to concrete disintegration. Vibration of the concrete at joints is not always a positive corrective measure. One procedure now being tried is to restrict the dumping from the buckets of the concrete mixer to some distance from the joint, and to place the concrete at the joint with hand shovels.

Too much finishing or manipulation of the concrete to obtain a good riding surface may lead to scaling. This would be aggravated in concrete showing a larger water gain, in that, low areas on the surface would be filled with sloppy mortar, which would eventually lead to scale.

CONCLUSION

Detailed investigation of a technical nature will have to be made in order to determine the actual cause of what might be termed the lack of durability of concrete roads in certain areas of the state. The probabilities based on some general studies are that numerous factors will be involved. Reasons are not clear or obvious, if they were, the causes of pavement failures would have been determined years ago, as considerable study has been given to this problem.

If aggregates, or combination of aggregate and cements, are found to be factors, corrective measures will have to be considered on an economic basis. In other words, the cost of corrections, such as importing aggregates, using special cement and subgrade conditioning, may prove to be a more costly procedure than to recognize the fact that concrete roads as built in these areas vary in life service.

This latter conclusion relates not only to concrete. We all have observed faster disintegration of natural rock ledges in some areas than in others. In recognition of this we may have to open our eyes

to the fact that if this exists in natural products we cannot expect any greater durability in man-made combinations.

Discussion to close in Supplement to be issued with Sept. 1939 JOURNAL, *should reach A. C. I. Secretary by June 1.*

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Durability of Pavement Concrete—Experience in Connecticut*

By E. C. WELDEN†

SYNOPSIS

Connecticut practice in the construction of concrete pavements has seen many changes, both in design, and methods of construction. Since the early twenties coarse and fine aggregates of good quality have been readily available. The use of a gravel or stone base under the pavement, and also the minimum amount of water necessary for workability of the concrete, has had much to do with the relatively good condition of the pavement concrete. Expansion joints and fillers in many cases have proved disappointing. Experience to date indicates that the use of vibrators with mixes of the proper consistency will improve the quality and durability of pavement concrete.

IT has often been said that the value of a pavement is measured by the service it will give over a period of years. This is as true today as when first stated. Concrete pavements are no exception and since they constitute a greater mileage than any other high class type of road surfacing in use today, it is only natural and fitting that the results obtained by the various state highway departments should be made available to the engineering profession through the American Concrete Institute. By careful study of information derived from years of practical experience and technical research, many of the earlier mistakes may be avoided, and capital invested in pavement surfaces will become an asset rather than a liability for those upon whose shoulders rests the burden of meeting the payments of maturing bond issues.

Concrete mixes, properly designed and fabricated, should produce an enduring material for either pavements or structural work. While

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in general the design of concrete is controlled by basic principles, simple though exacting, there are many features to be considered in the actual design of pavements and in the selection and proportioning of aggregates, in order that the results attained may provide the proper balance between the capital investment and the expected service life of the pavements to be constructed. Good engineering does not justify the expenditure of money in excess of an amount necessary to produce a well balanced design.

In reviewing the experience of the Connecticut State Highway Department, it is interesting to note, that some of our earlier concrete pavements, constructed in 1914-15, when the water cement ratio was an unknown quantity, and the proportion of fine to coarse aggregate was controlled in wheelbarrows by none too careful workmen, are still functioning and give every indication of many more years of useful service. Though there are numerous cracks in these pavements, both longitudinal and transverse, and settlement at some of the joints, there is very little indication of deterioration despite the heavy traffic to which the pavements have been subjected, through summer heat and winter cold, for nearly a quarter of a century.

GOOD DRAINAGE, GOOD AGGREGATES

While it has often been said that the first requirement for a good road is proper drainage, it is equally true that a structurally sound concrete pavement can be obtained only by the selection of proper aggregates, combined in proportions with a minimum water-cement ratio, that will insure satisfactory service and durability for the expected life of the pavement. This, I believe, accounts to a great extent for the exceptionally good condition of some of the earlier pavements to which I have referred. On these particular sections, drainage conditions were good, and the aggregates were of exceptional quality, the coarse aggregates from local ledges of trap or granite, and the fine aggregates being clean silica sand obtained from Long Island Sound.

The practice in Connecticut, in the design and construction of concrete pavements, has followed in general the methods of other states in the northeastern group. Blessed with an unlimited supply of trap rock, we have had very little to worry about in the selection of a uniform, durable coarse aggregate. Several modern crushing plants produce excellent materials at reasonable prices, and are available for rail, water or truck delivery. For several years satisfactory control of the fine aggregate was lacking, there being no local plants equipped to supply a processed material that would consistently meet standard specification requirements. Suitable sands were first obtained

from Long Island but transportation costs prohibited their general use except in localities adjacent to water delivery.

SCALING AND CRACKS

To say that all of our concrete pavements have proven satisfactory can hardly be justified by the service record of some, although I do not wish to imply that we have had a complete failure of any section of concrete road. There have been cases of surface scaling and many cracks where subgrade conditions were poor. With few exceptions scaling has been light and has not been followed by disintegration of the concrete underneath. Many of these roads have been surfaced with bituminous materials, more to improve their appearance than for lack of ability to function under traffic as originally designed. However, I believe that in many cases we have been able to establish the reasons for these failures and to profit by such knowledge in our design of more recent pavements.

The realization of the inherent limitations of concrete to withstand the complex stresses to which pavement slabs are subjected, from heavily loaded vehicles and rapidly changing temperatures, makes it imperative that every effort be made in the design and preparation of the subgrade, to assist the concrete to resist these stresses. Since uniformity of subgrade in Connecticut seldom exists for any considerable distance, we have found it necessary to supply from 12 to 24 inches of gravel, or stone, to form a more nearly adequate support for the concrete slabs. Without doubt, this added expense has been many times justified in reducing the tensile stresses to which the concrete in the slab is subjected, thereby prolonging the life of many of our concrete pavements.

WIDTH AND CROSS-SECTION

The design for our earlier mixed concrete pavements was a 6-8-6-in. section, 16-ft. width, with joints at 25 ft. intervals, the mix being 1-2-4 by guess or by volume. While the earlier construction lacked the present refinements, much of it is still functioning either in its original section or as a base for a bituminous top. In the development of the highway system, however, it has been necessary to replace some of the original sections, because of their inadequate width and alignment, with concrete of more modern design.

It was only a short time before the 16 and 18 ft. sections gave place to 20 ft. roads built in two parallel lanes, opening each lane to one-way traffic as soon as the concrete developed the required strength of not less than 500 lb. as indicated by transverse tests. The time required to obtain this strength depended on seasonal weather conditions and the brand of cement used, but this strength was usually obtained in

from 7 to 8 days. I believe that Connecticut was one of the first, if not the first state to use the parallel lane method of construction. It might be said that this method of construction was adopted in order to eliminate the necessity of detouring traffic, since at that time there were few parallel roads capable of providing for the then heavy traffic of industrial Connecticut.

While the earlier concrete pavements were built to a 6-8-6-in. section on a flat subgrade, the design was soon changed to uniform sections ranging from 7 to 10 in. thick, depending on the density and character of the traffic to be served. We are still using a uniform cross section usually 8 in. thick, never having adopted the thickened edge design.

CURING

Methods for curing our earlier concrete pavements included the use of earth, hay, straw and burlap, kept wet for from 48 to 72 hours varying to some extent with seasonal conditions, but the amount of water actually used was more often influenced by the physical condition of the man at the end of the hose.

Water is still being used as the curing agent, retained by mats made with a cotton or shoddy filler between burlap covers. Since additional water is necessary for proper hydration of the cement, the method by which moisture is made available is of less importance than the assurance that it will be supplied as often as needed. Fortunately both mechanical and hand operations are now more carefully controlled and the matter of proper curing is no longer left to chance.

The use of calcium chloride as an admixture, in a solution containing 2 per cent of chloride by weight of cement, was standard practice for several years. Its use was not intended as a curing agent but rather to accelerate the hardening of the concrete, that the pavements might be opened to traffic sooner than would otherwise have been possible. Calcium chloride in excessive amounts, applied to the surface of the pavement, either as a curing agent, or for ice removal, should be avoided since scaling may follow.

REINFORCED SLABS

We are apt to think of pavement concrete in terms of compressive strengths, rather than its ability to withstand the bending stresses produced by moving loads. Such stresses are especially severe at the edges and corners of the slab. Since the compressive strength of concrete is approximately six times its tensile strength, failures usually result from lack of resistance to tension rather than lack of ability to withstand compression, to which it may be subjected. Tests on experimental sections of pavement indicate that the unit stresses at rupture

should be at least twice the unit stresses to which the pavement may be normally subjected if failure is to be avoided.

To obtain greater tensile resistance, reinforcement of either the bar or mesh types has been used in nearly all of our concrete pavements. The amount of steel per 100 sq. ft. has varied with the thickness of the pavement and type of reinforcement used. Light mesh was first used, sometimes placed two inches from the top and sometimes two inches from the bottom of the slab. For several years we used only marginal bars, two inches from the top and bottom of the slab, and three inches from the edges of the slab with two 5-ft. radial bars at the corners two inches below the surface of the concrete. While it would be hard from a technical point to justify the unequal distribution of the steel, it has proven satisfactory and there seems no logical reason for having changed back to the mesh type of reinforcement, except that a uniform distribution of steel tends to hold together the fractured surfaces of the concrete at transverse or longitudinal cracks.

JOINTS

The property of concrete to expand and contract with changing temperatures has long been recognized in the design of concrete pavements. On our earlier work the spacing of expansion joints varied from 25 to 100 ft. with no provision to predetermine the location of contraction cracks. Transverse cracks developed at more or less regular intervals. Though unsightly, these cracks caused very little trouble from a maintenance standpoint and did not impair the riding qualities of the pavement.

Each slab was designed as a separate unit, no provision being made for load transference from one slab to another. Experience in the field soon indicated this line of reasoning to be unsound and necessitated the consideration of some method for load transfer at expansion joints. While different methods were used to accomplish this purpose, a closely fitting cylindrical steel pin, sliding in malleable iron castings embedded in the ends of the concrete slabs, proved fairly satisfactory when spaced at 2 ft. intervals across the expansion joint. Since their adoption there has been much less trouble from settlement or failure of the concrete at the joints.

Present practice provides for expansion joints at 75 ft. intervals with two intermediate dummy joints at 25 ft. intervals. There seems to be no logical reason for carrying reinforcement continuously across dummy joints which are placed in the pavement to act as contraction joints. Any contraction joint, or expansion joint, to function so as to relieve tensile stresses, must necessarily be free to move. Dowels at dummy joints have been omitted, depending on the interlocking

of the coarse aggregate for whatever bond there may be to keep the slabs in line. While this may not be standard practice there has so far been no indication of failure.

While we have used many types of joint fillers, both premolded and of the air-cushion type, none has proved entirely satisfactory. The ideal filler must be non-extruding and of a material that will remain at all times in intimate contact with the concrete throughout the range of temperatures to which a pavement may be subjected.

For sometime we have been using a pre-compressed cork filler, sealing the top 2 in. of the joint with latex or a bituminous material. Some of these materials give promise of satisfactorily sealing the joints under wide temperature changes.

HASSAM PAVEMENT

A history of the concrete pavements of Connecticut would not be complete without reference to Hassam concrete pavement, about 70 miles of which was laid in the State from 1907 to 1923. The first section, on Methodist Street, City of New London, in 1907, is still in use and shows every indication of added years of service. The same may be said of many other sections of this type of pavement laid during the years following 1907.

Nearly all of the Hassam pavements were laid to a uniform thickness of 6 in., using 2-in. trap rock as the coarse aggregate. The stone was spread by hand and thoroughly compacted with mechanical rollers. The voids, which seldom exceeded 35 per cent, were then filled with a creamy grout made up of 1 part portland cement to 1½ parts sand. After all voids had been completely filled with grout, the pavement was allowed to set for from 30 to 60 minutes, after which the roller was again used, bringing to the surface the excess water that flowed to the shoulders of the road, leaving a solidly compacted, dense concrete.

The cost of this pavement was somewhat more than a mixed concrete since more cement and more stone were required to the square yard—a 6-in. pavement requiring about one bag of cement to the square yard.

I doubt very much if the promoter of Hassam concrete, in his search for a cheap durable pavement, ever thought that the engineers of to-day would be striving for the same result by a method almost directly opposite to that on which his patent was based. In the one case excess water was used to insure that all voids were filled with grout, the excess water beings queezed out by rolling, while present practice uses only enough water to provide workability.

The Hassam patent expired in the early twenties but there are still in service in New England many sections of this type of pavement that testify to the enduring qualities of this type of construction.

FREEZING AND THAWING

Concrete is not an impervious material, and may be adversely affected by the presence of water when subjected to alternate freezing and thawing. Since durability is the most essential quality to be attained in a concrete pavement, every effort has been made to eliminate, as far as possible, all factors tending to contribute to unsatisfactory results. The fact that workability is an essential quality for proper placing and finishing, has led to the use of excess water, resulting in a low strength concrete of higher absorption and of questionable durability. The importance of a low water-cement ratio and the minimum cement content consistent with strength desired, cannot be emphasized too strongly when durability and permanence are to be the governing factors.

VIBRATED CONCRETE

While internal vibration of concrete in all bridge structures has been our standard requirement for sometime, it was not until 1936 that vibration was required in the construction of concrete pavements. Previous experiments with structural concrete had shown an increase in strength, as well as other desirable qualities, through the use of vibrating equipment and a reduction of the water-cement ratio.

Two types of vibrators have been used, the vibrator screed and the pan type between the screeds. The tubular type of vibrator was tried with only partial success, since there was not sufficient power to move the vibrating unit through the mass.

Unless one realizes the principles involved in vibrating concrete, the results may prove to be disappointing. Excess water and oversanding produce separation of the coarse and fine aggregates, defeating the purpose and benefits for which the method was intended. Results obtained to date indicate the desirability of further study for the development and adaptability of this type of equipment to realize all of its possibilities.

Since we adopted the use of vibrators as standard equipment on paving contracts, no difficulty has been experienced in finishing a relatively harsh stony mix, when the finishing machines are amply powered and in good mechanical condition. The consistency of the mix that may be successfully vibrated, is controlled more by the efficiency of the mixing and placing equipment to handle a relatively dry concrete, than by the ability of the vibrator properly to compact concrete of a given consistency.

Increased effectiveness of high frequency vibration seems to have been amply demonstrated when compared with the results obtained with low frequency units. A minimum frequency of 5500 r.p.m.

maintained at a constant speed and constant amplitude should give excellent results when used with a properly proportioned mix.

MIX DESIGN

Too often our concrete mixes have been proportioned on the basis of compressive strength, without proper regard for the requirements of durability and economy. While sufficient strength to carry the load is absolutely essential, resistance to the elements as expressed by durability is of equal, if not greater importance.

Having in mind the requirements of durability, strength and economy, various mixes were tried in the laboratory to determine workable proportions that would give the required strength and durability with a minimum cement content. As a result of these studies, a 1-2.4-4.7 mix, by weight of cement, was adopted as standard. Using a maximum of 6 gal. of water per bag of cement, the total volume of concrete produced would be 5.08 cu. ft; requiring for one cubic yard of concrete 1.33 bbl. of cement; .44 cu. yd. of sand; and .83 cu. yd. of stone; the sand used being 34 per cent of the aggregate.

Very satisfactory strengths were obtained with this mix, although any change in the grading of the coarse aggregate was reflected in the workability of the concrete.

Our specifications required the use of two sizes of stone combined in a 70-30 ratio at the batching plant, approximately 100 per cent of the maximum size passing 2½ in. square openings, and approximately 100 per cent of the smaller size passing a ¾-in. square opening.

In a few cases cores cut from the pavement indicated a small amount of honeycomb at the reinforcement when the mats were placed 2 in. from the bottom of the slab. Trouble was also experienced when the reinforcement was placed 2 in. from the top of the slab, since large stones resting on the steel would tend to raise the finishing machine from the forms. To overcome this difficulty, the maximum size of the coarse aggregate was reduced so that approximately 100 per cent passed 2-in. square openings. This change increased the voids in the coarse aggregates about 2 per cent, to a total of 44½ per cent. Both fine and coarse aggregates were slightly decreased, resulting in a 1-2.2-4.4 mix. This gave a cement factor of 1.42 bbl. of cement to the cubic yard of concrete, when using 5.5 gal. of water to the bag of cement, this being the maximum allowed.

To obtain information relative to the durability of the concrete in our more recently constructed pavements, cores were cut from various sections and tested for compression, absorption and density. These tests were made according to standard methods prescribed by the American Society for Testing Materials.

The average compressive strength per square inch for the cores cut from pavements (averaging less than three months old) was 6300 lb.

To study the uniformity of the concrete at various depths in the pavement, the cores were cut transversely into three equal sections and the average percentage, by weight of water absorbed, at the different depths, was obtained. Average absorption, by weight of water, for the top sections of all cores was 3.47 per cent; for the middle sections 3.13 per cent, while the lower sections showed an average of 2.96 per cent. The difference in absorption between top and bottom of cores may be the result of too much vibration, since from visual inspections there appeared to be somewhat more mortar in the top than in the bottom of the cores.

Specific gravity and density tests *showed a specific gravity range of from 2.515 to 2.588 and a density range of from 88.3 per cent to 90.7 per cent. No attempt should be made to draw conclusions from the limited number of tests.

The use of portland cement conforming to the standard specifications of the American Society for Testing Materials has been standard practice for all of our paving contracts. No attempt is made to secure a product from any particular mill or location. The condition of the pavement concrete indicates that equally satisfactory results have been obtained with cements from either the Lehigh Valley or Hudson River Districts. Our experience indicates the advisability of leaving to the manufacturer the preparation of the specification under which his product is to be produced, with more attention on the part of the engineer to the selection of proper aggregates and the mechanical operations incidental to mixing and placing of the pavement concrete.

Rational planning and design of a concrete pavement is an engineering and economic necessity. Development of the highways has not kept pace with the demands of traffic. As a result there is need to rebuild many miles of road that can no longer be said to serve adequately the traffic of to-day. Since it is not unreasonable to assume a service life of 30 years, or more, for a modern concrete pavement, both design and construction methods should be based on probable future needs of traffic if the highway dollar is to serve the purpose intended.

During the last 25 years there has been a gradual and constant change in the design and construction methods of Connecticut's

*Per cent of density was obtained by dividing the weight of a given volume of hardened concrete by the theoretical weight of solids in the same volume as calculated from the proportions and specific gravities of each of the constituent materials.

concrete pavements. The trend has always been toward the selection of materials and equipment that would produce a concrete worthy of the slogan—"Concrete for Permanence."

Discussion to close in Supplement to be issued with Sept. 1939 JOURNAL, *should reach A. C. I. Secretary by June 1.*

Vol. 35 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

7400 SECOND BOULEVARD, DETROIT, MICHIGAN *APRIL 1939*

Job Problems and Practice

Some will ask questions — Some will answer them — Some will do both

A. C. I. members are invited to use this new JPP department as an informal means toward mutual help. When a problem holds possibilities of general interest the discussion will be briefed in these pages.

If you have a problem—present it; a question, ask it. If you know the answer to a question asked, or if you can contribute something which may help in the solution, or have reason to disagree with the answers or suggestions published, your contribution will be welcome.

If you know of an interesting problem whose solution has already smoothed the way of someone in the field, tell us about it—some other A. C. I. member may need just that information.

The "answers" are the answers of individuals to whom the questions are referred and not of the A. C. I. as an organization.—EDITOR

What heat will concrete withstand without deterioration? (No. 35-28—Feb. '39, p. 292)

A—There is no question that portland cement concrete which will have a satisfactory life can be made for these temperatures.* The most important consideration is the selection of aggregates. Undoubtedly Haydite or some other form of burned clay aggregate will eliminate some of the possibilities of trouble. High silica materials should be avoided since the expansion of these materials may cause the concrete to crack and spall.

Our experience with concrete exposed to continuous heat has been largely for temperatures above 500° F. or 600° F. since portland cement is satisfactory up to these temperatures. Some of the statements made regarding the effect of heat have not been borne out by our tests and experience when high alumina or fused cements such as Lumnite are used. For example, concrete heated to 1000° F. for a

*300° to 500° F.

few days will have no more strength than concrete heated to 1000° F. for five years. Neither the duration or the quantity of heat units affect the strength of Lumnite concrete except as they affect the temperature. The strength of concrete heated to 500° F. or higher will have approximately the same strength whether the cold strength is 2000 or 5000 pounds in compression. It is impossible to dehydrate Lumnite completely at 1000° F. regardless of the duration of exposure. We suspect the same is true of portland and that the reason for its failure at this temperature is expansion or growth and not dehydration of the cement.

It is our opinion based on our field observations that the products of hydration of portland cement have a continued growth or expansion when heated to temperatures higher than 500° F. to 600° F. over long periods which causes failure while the products of hydration of Lumnite are practically neutral to volume change under the same conditions.

Perhaps the following test results will be of interest:

Compressive Strength, lbs. per sq. in. of High Alumina Cement Concrete after Exposure to Various Temperatures

Normal	1000° F.	1250° F.	1500° F.	1750° F.	2000° F.
1895	757	919	343	652	1699
2253	592	no test	369	822	1075
2617	no test	no test	446	228	502
3125	no test	no test	488	157	351
		1100° F.	1400° F.	1700° F.	
7070		993	775	452*	

*Heated for 4 hrs. only. All other specimens heated six days.

The difference in strength above 1500° F. is a result of the development of fired strength which varies with the nature of the aggregate used.—R. T. GILES*

Must wire mesh reinforcement be free from rust?
(No. 35-32)

Q—Should cold drawn wire mesh reinforcement be kept free from rust when it is to be used in concrete pavement slab?

A—No. The wire should be allowed to rust, at least slightly, in order to provide bond. Cold drawn wire has a very uniform, smooth surface and we have succeeded in slipping two feet of such wire (new) through good quality concrete, shearing two cross wire spurs, whereas the same wire bonded well when permitted to rust even superficially.—F. V. REAGEL.†

*Service Director, Atlas Lumnite Cement Co., New York.
†Engineer of Materials, Missouri State Highway Dept., Jefferson City.

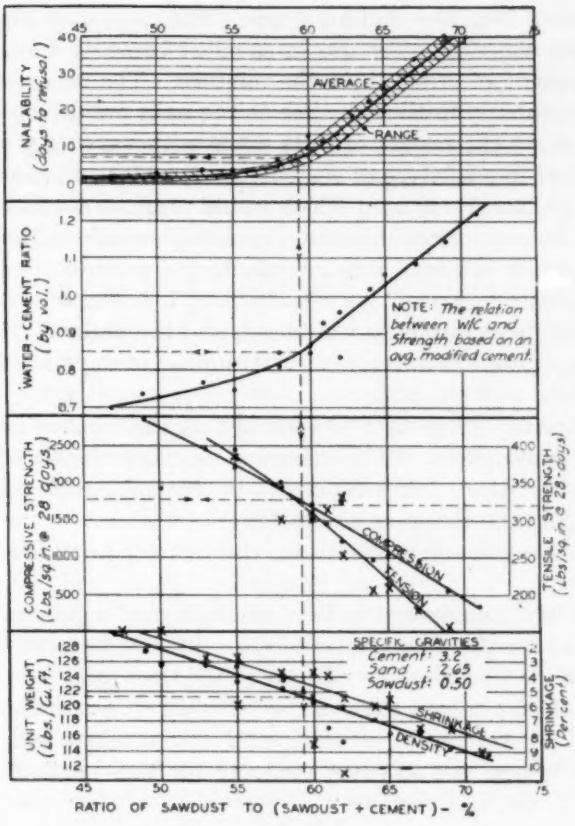

FIG. 1—RELATIONSHIP BETWEEN DIFFERENT PERCENTAGES OF SAWDUST IN SAWDUST CONCRETE AND NAILABILITY, W-C RATIO AND DENSITY

Sawdust (nailing) concrete (No. 35-33)

Concrete into which nails can readily be driven for the purpose of holding roofing material, flashing, etc. in place is referred to here as nailing concrete. There are various patented processes such as absestos fiber, cinders, foaming agents and wood sawdust, but this discussion is limited to wood sawdust concrete.

The problem is to obtain a mixture which will be sufficiently strong from a structural standpoint and at the same time able to take nails during the period in which it is planned to drive them. The concrete should consist of cement, sand, pine sawdust and sufficient mixing water to produce a slump of about 2 in. The sawdust should be clean and of such size that it passes a ¼-in. square opening screen and

retained on a No. 14 standard screen. Hickory, oak, birch or cedar sawdust are unsuitable for use in sawdust concrete from standpoint of development of strength in the mixture. The concrete should be cured continuously until such time as the nails are driven into it.

Fig. 1 shows the results of tests made with varying percentages of pine sawdust in a mixture of cement, sand, water and sawdust. That proportion of sand was used which would produce a workable mix for the water-cement ratios indicated. It may be seen from the figure that, as the ratio of sawdust was increased, the water-cement ratio and nailability were increased, accompanied by a decrease in strength and density. Eight-penny nails were used, and the nailability of the mixtures were evaluated in terms of the number of days after placement each would accept the nails.

In conducting the tests it was found that the amount of sand could be varied considerably without having any appreciable effect on the nailable properties of the concrete; in general, sufficient sand was added to hold the slump at about 2 in.

The following example illustrates the manner in which Fig. 1 may be utilized:

Assume that it is desired to have (1) A 28-day compressive strength of 1000 p.s.i. in the "nailing concrete"; (2) A density of 115 lbs. per cu. ft.; and (3) Considerable latitude in the time limit for nail driving.

A ratio of 65 per cent sawdust to (sawdust + cement), or a mix of 1:1.75:1.75 by volume of cement, sand and sawdust will meet these requirements and the sawdust concrete for this particular mix will accept nails for approximately 3 weeks after placement—JOHN C. SPRAGUE*.

Sulphur for capping test cylinders? (No. 35-34)

Q—Will you supply some information as to the use of sulphur for capping test cylinders?

A—The practice of using sulphur in conjunction with other materials, such as fine clay, limestone dust, sand, etc. for capping purposes has been followed for some time in many laboratories. The usual procedure is to mix sulphur and some other powdered material in equal parts, melting and pouring caps on the ends of the test specimens, employing a suitable mold for this purpose. The sulphur mixture hardens on cooling and gives a cap of high strength. The important requirement is that the ends of the capped specimens be plane and parallel.

Capping machines (molds) are manufactured by a number of different concerns. They employ two parallel machined plates held, by

*Chief Concrete and Soils Section, U. S. Engineer's Office, Huntington, W. Va.

some suitable device, at a distance such as to give caps of desired thickness. The usual method is to place the cylinder horizontally between the plates. The holding device is provided with openings so that the molten mixture can be poured into the narrow space between the cylinder ends and the plates.

It has been called to our attention that at Fort Peck Dam 1½ per cent of activated charcoal is used in the mixture which seems to absorb some of the gases generated.—A. G. TIMMS.*

Safe length of haul for ready-mixed concrete? (No. 35-35)

Q—What is the effect of time of haul on ready-mixed concrete?

A—There have been no recent investigations of a formal nature. Most of the studies along these lines were prior to 1930. At that time present-day practices had passed the development stage and the data should be as significant now as then. The principal question to be raised with respect to them is that of whether or not cements have changed sufficiently in the past ten years to have an important effect on the relationships. I doubt if cements have changed enough to destroy the significance of the earlier data.

A comprehensive investigation of Jaeger truck mixers was made during the summer of 1931 by S. C. Hollister. The report of his tests is published in this JOURNAL in 1932[1] and (in greater detail) in a pamphlet issued by the Jaeger Machine Co. One of the Mr. Hollister's conclusions in his A. C. I. report is:

> So long as the mix remains workable there appears to be no variation in strength for mixing periods varying from 40 revolutions of the drum to 1½ hours of mixing. Tests were not conducted beyond the latter period.

From a graph in his more detailed report, the following typical results were taken for concrete containing 8.2 gal. of water per sack of cement and 5 sacks of cement per cu. yd. of concrete, mixed in a 2½-cu. yd. mixer rotated at 6.8 r. p. m.

Time of Mixing—Minutes	9	20	40	60	90
Slump—Inches	7.5	8.0	6.5	7.0	4.5
Compressive Strength—28 days	3100	3100	2750	3150	2900
Compressive Strength— 7 days	2150	2150	2200	2350	2250

I selected a fairly high slump mix in order that the differentials might be more clearly shown. Data are also given for lower slumps. For example, a mix containing six sacks of cement per cu. yd. of concrete and 6.02 gal. of water per sack showed slumps varying from about 2½ in. at 40 revolutions to about 1 in. at 60 minutes.

While the reduction in slump shown in the preceding tabulation is insignificant up to 1 hour, and not great at the end of 1½ hours, it should be pointed out that this test reported conditions which are

*Portland Cement Assn. Research Laboratory, Chicago.

[1]JOURNAL, Amer. Concrete Inst., Feb. 1932; *Proceedings* Vol. 28, p. 405.

probably more severe than would be encountered in modern practice.[2] Modern practices fix "mixing speed" as being in excess of 4 r. p. m. and "agitating speed" as from 2 to 6 r. p. m. It limits the number of revolutions at mixing speed to a maximum of 150 and requires that additional revolutions be at agitating speed. Consequently, the likelihood of obtaining as many as 612 revolutions in 1½ hour (6.8 x 90) is remote. It seems reasonable to observe that the number of revolutions had as important an effect as did the time.

The late Prof. W. A. Slater made tests of an agitator body manufactured by the Clinton Motors Corp.[3] and describes tests of concrete transported for periods varying from a few minutes to 2½ hours. With respect to these tests Professor Slater stated:

> The stiffening which occurred within the first hour would improve the consistency of the concrete for almost any purpose. Even after 2½ hours in the drum, the consistency would be quite satisfactory for road and pavement work where the requirements are quite rigid.

For an average of 4 series of tests Professor Slater showed that concrete having an initial slump of about 7½ in. was reduced to a slump of about 5 in. after about 40 min. and remained at about that level until 70 min.; at 160 min. the slump had reduced to about 2½ in.

Tests carried out under the direction of Miles N. Clair, in behalf of the Boston Concrete Corp. were reported by the National Ready Mixed Concrete Association[4]. The results show that the concrete strengths are practically unaffected, or slightly increased, by periods of agitation up to about 2 hours and that the decrease in slump for a period of agitation up to about 1 hour is small enough as to be considered negligible under average conditions.

As early as 1921, the U. S. Bureau of Public Roads made tests on "Central-Plant-Mixed Concrete Tested for Maximum Safe Haul"[5]. Concrete was hauled in an ordinary truck body and sampled at various periods up to about 3 hours. While a decrease in plasticity was found, the strengths showed practically no change.

"Report of Field Tests of Concrete Used on Construction Work" by Slater and Walker,[6] includes a limited amount of data bearing on this question. Concrete allowed to stand ½ hour before molding the specimens gave 28-day strengths about 25 per cent greater than obtained from specimens molded immediately after discharging the concrete from the mixer. Little further increase was obtained for tests of specimens molded 1 hour after mixing.

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[2] See Tentative Standards and Recommended Practices for Truck Mixers and Agitators of Revolving Drum Type, National Ready Mixed Concrete Association.
[3] *Proceedings*, A. S. T. M. Part II, 1931.
[4] April 23, 1931.
[5] *Public Roads*, Dec. 1921, p. 22.
[6] *Proceedings*, A. S. C. E., Jan. 1925.

Gonnerman and Woodworth in their "Tests of Retempered Concrete"[7] report tests of concrete allowed to stand for periods up to 6 hours and protected from evaporation which gave essentially the same strengths. When the concrete was allowed to stand unprotected, so that water was lost through evaporation, an increase in strength resulted up to the time when the concrete ceased to be plastic.

Tests by the J. L. Shiely Co., St. Paul, Minn.,[8] show comparisons between the strength of concrete sampled at the plant and sampled after being hauled for 12 miles in an agitator truck. The average of 3 trips showed the strength of specimens taken after the haul to be about 5 per cent higher than of those taken at the plant. There was relatively little decrease in slump.

While there is voluminous literature on the subject of ready mixed concrete, (a large proportion of which we have listed) I believe that the foregoing references are to data which are typical. Based on such data, the specifications of the American Society for Testing Materials and the recommendations in the Progress Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete provide for transportation periods up to 1½ hours. With respect to that limitation, the Joint Committee (Section 308) is as follows:

308—*Period of Hauling Ready Mixed Concrete.*—In Sec. 315-S, a period of 1½ hr. within which delivery of ready-mixed concrete must normally be made is specified. This is based on the results of research which have shown that a period of 1½ hr. between the beginning of mixing and the completion of the discharge of the batch results in no detrimental effects on the concrete. The data indicate that under specially favorable conditions periods up to 2 and 3 hr. may be allowed. Experience has shown that with certain soft aggregate greatly prolonged mixing will result in a decrease in size of aggregate particles and a corresponding increase in the proportion of fine material. When air temperatures are unusually high, or the ingredients of the concrete mixture are such that an unusually quick time of set or loss of plasticity may occur, it may be necessary to specify periods shorter than 1½ hr. to insure that no damage be done to the concrete. The footnote in the specifications suggests limits which might be used under these conditions.

—STANTON WALKER*

Measuring aggregate on small job? (No. 35-36)

Q—What would be the most accurate method of proportioning concrete aggregates on a small job when weighing equipment is not available?

A—A simple but satisfactory way of batching aggregates is to place them in a bottomless measuring box, conforming to the pan of

[7] *Proceedings*, Amer. Concrete Inst., Vol. 25, p. 344.
[8] Nat'l Ready Mixed Concrete Assn., 1931.
*National Ready Mixed Concrete Assn, Washington.

the wheelbarrow. The box is filled, struck off, and lifted from the barrow giving a uniform amount of aggregate in each batch.

Leaky masonry walls (No. 35-37)

Q—There seems to be an unwillingness to discuss the problem of leaky masonry walls. The problem is serious and common. Suggestions in the way of a remedy seem to touch only the edges of the subject. I fail to find anything in the limited literature dealing with *curing the walls.* If curing is good for concrete in general why isn't it good for masonry walls in which the mortar joints seem to be the seat of much of the trouble. It is my belief that the rapid withdrawal of the moisture of the mortar into the wall units, whether brick or concrete block, leaves a soft mortar and contributes a good deal to the condition of interior dampness. I should like to see a thorough discussion of the matter.

Non-shrink grout and concrete (No. 35-38)

In connection with foundation and contraction joint grouting the question arose as to how long grout could be mixed and still be suitable for use.

A first batch of neat cement with a w/c of 1.00 by volume, using the Coulee Dam modified cement, was mixed 11½ hours, and sampled every half hour. 7, 28, and 90-day, 2 by 4-in. cylinders were made, for each age sampled. As a result shrinkage before initial set was reduced 90 per cent (from a settlement of 22½ per cent to a settlement of 2½ per cent.)

Seven-day strengths were decreased 70 per cent with 11½ hours mixing, but 28-day strengths were reduced only 47 per cent, and 90-day strengths only 17 per cent with that long mixing.

With a high early strength cement and a w/c of 1.2 by volume, shrinkage was reduced 95 per cent in 7 hours mixing, with no definite loss in 90-day strength.—O. G. PATCH.*

*Engineer, Bur. of Reclamation, Coulee Dam, Wash.

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Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Liquid oxygen blasting in clay quarries

G. PFISTER, *Tonindustrie Zeitung*, Vol. 63, No. 7, p. 70 (Jan. 23, 1939) Reviewed by A. U. THEUER

This article describes in detail methods of preparation and of placing liquid oxygen-cartridge charges for moving clay embankments. Advantages are claimed for its use in cities or localities where disturbances must be sharply circumscribed.

Concerning the transverse stability of the Rohrback bridge

R. BUSEMANN, *Beton und Eisen*, Vol. 37, No. 24, p. 394 (Dec. 20, 1938)
Reviewed by A. U. THEUER

Included in this article is a study based on model analysis of the rigidity of a reinforced concrete arch subjected to transverse wind loads. This article is one of a series of studies that were carried out in conjunction with field tests on an actual arch.

A new method of pouring hollow ribbed concrete decks

K. KUTSCH, *Beton und Eisen*, Vol. 38, No. 2, p. 36 (Jan. 20, 1939) Reviewed by A. U. THEUER

A unique method of pouring hollow ribbed concrete decks by the substitution of inflated tubes for ordinary form work or filling is briefly described. The inflated tubes are laid out on a base slab between successive lines of steel rib reinforcing. The deck and ribs are then poured simultaneously. After one day of hardening the tubes are deflated and withdrawn for use elsewhere. A saving in either form lumber or filling material as well as a saving in labor is claimed for this method of construction.

Vibrations in concrete water tanks

DEAN S. CARDER, *Engineering News-Record*, Vol. 121, No. 24, Dec. 15, 1938, pp. 767-768.
Reviewed by. S. J. CHAMBERLIN

Tests on an elevated concrete water tank at Sacramento showed a dominant vibration period of 0.29 secs. for all ranges of water height. Observations at some points gave slightly different periods, probably because of local disturbances. Amplitudes varied with the height of the structure. Tests on steel tanks have shown large differences in periods of vibration for full and empty conditions.

Physical and mechanical characteristics of soils

M. BUISSON, *Le Constructeur de Ciment Arme*, Vol. 20 and 21, April, 1938 through January, 1939, and continuing Reviewed by P. H. Bates.

Starting with the April, 1938, issue and continuing (not completed) in the January, 1939 issue is a series of articles covering soils by Buisson. All of the customarily used methods of studying soils have been covered by the author. Indeed, he has included a number of unusual approaches to this subject. Description of the methods, discussion of the data and their significance, are given by the author.

Designing workable concrete

C. A. G. WEYMOUTH, *Engineering News-Record*, Vol. 121, No. 26, Dec. 29, 1938, pp. 818-820.
Reviewed by S. J. CHAMBERLIN

Different sized coarse aggregates were combined to give workable concrete for various conditions of placing without increasing the water-cement ratio. Mortar-voids curves, made from laboratory studies, were used in designing the mortars. Gradings of the coarse aggregate as delivered were obtained and the different sized aggregates combined to prevent particle interference.

Reinforced concrete shells

Bautechnische Mitteilungen, Berlin, No. 2, pp. 5-8, June-Sept., 1938. Reviewed by F. L. EHASZ

Possibilities of application and advantages of the various forms of shell structures are pointed out, together with some notable examples as illustrations. To the development of reinforced concrete may be attributed the advances made in the shell form of construction. The Zeiss-Dywidag system is referred to as the foremost pioneer in this field.

Testing for deleterious materials in concrete aggregates

F. C. LANG, Minnesota Highway Department, Highway Research Board.
HIGHWAY RESEARCH ABSTRACTS

This paper describes a method of measuring the harmfulness in concrete of certain deleterious materials in the coarse aggregate. Small concrete beams were made in which the coarse aggregate was entirely composed of one kind of deleterious material. The beams were tested in flexure and were also subjected to freezing and thawing. This appears to be a better method of testing than freezing and thawing the aggregates separately.

The construction of the Eichelberg bridge

HANS RIETLE, *Beton und Eisen*, Vol. 38, Nos. 1 & 2, pp. 1 & 25 (Jan. 5 & 20, 1939)
Reviewed by A. U. THEUER

In great detail the special geological conditions and esthetic considerations that led to the adoption of a concrete girder bridge in place of an open cut and fill to carry the German East-West superhighway over a summit in the Swabian Alps is described. The structure as adopted consists of 41 bents of varying span representing a total length of 3080 ft. Part of the structure is on a 7 per cent grade with a 1300 ft. radius. Both the structural design as well as special details and methods adopted in construction are discussed.

Magnesite as a cement for massive concrete

GRUN and SCHLEGEL, *Zement*, Vol. 27, No. 52, Dec. 29, 1938, pp. 817-819.
Reviewed by L. T. BROWNMILLER

A series of tests on specimens of magnesite cement is reported in this article. The effect of the concentration of the mixing solution, of the burning temperature at which the cement is produced, of the amount of mixing solution and of the proportion of sand were investigated. The tests showed very high initial strengths, but the cement was not water resistant. The addition of a puzzolanic material decreased the strength somewhat and also increased the resistance to attack by water.

Why concrete pavements curl

CLARAN F. GALLOWAY, *Engineering News-Record*, Vol. 121, No. 25, Dec. 22, 1938, pp. 800-801.
Reviewed by S. J. CHAMBERLIN

Roughness records (obtained from a profilometer) taken regularly on Los Angeles County pavements and compared with weather conditions indicate that the curl

in the slabs is due chiefly to the moisture-content differential between top and bottom. Shorter lengths of slab, due to cracking, gave less roughness indices. Well drained subgrades should make for less moisture differential in the slab and hence less curl at the joints.

The effect of sand on the durability of mortar

BERT MYERS, Iowa State Highway Commission, Highway Research Board.
HIGHWAY RESEARCH ABSTRACTS

This report shows the loss of strength due to freezing and thawing of mortars made from three brands of cement each combined with five sands of different lithological composition. Results indicate that the relative durability of cement as indicated by freezing and thawing tests of mortar will depend upon the kind of sand used, and that the relative durability of sand as indicated by the same test will depend upon the cement used.

Bicycle paths along highways

HANS SCHACHT, *Forschungsarbeiten aus dem Strassenwessen*, V. 12, pp. 1-60, 1938.
Reviewed by F. L. EHASZ

Almost 20 million people are estimated to travel on bicycles in Germany. Highways on which this type of traffic is extensive have been provided with paths, designed with a view to separating streams of bicycles from those of automobiles. Americans touring in Germany view with amazement the widespread use of this mode of travel by people going to and from work, school or recreational journeys. More than half of the space in this paper is devoted to photographs and sketches which show various types of pavements employed.

A study of map cracking in sand-gravel concrete pavements

W. E. GIBSON, Kansas State Highway Department, Highway Research Board.
HIGHWAY RESEARCH ABSTRACTS

Tentative conclusions reached in this report are: (1) The information up to date does not indicate that map cracking in sand-gravel aggregate pavements can be foretold by the chemical analysis of the cement; (2) This type of map cracking is predominantly occurring in sand-gravel aggregate concrete pavements in Kansas; (3) The autoclave test as now made on cements is not a criterion as to whether or not map cracking will develop in sand-gravel aggregate concrete, and (4) This disintegrating map cracking has been successfully reproduced in the laboratory.

An improvement in the method for measuring the shrinkage of highway cements

H. E. SCHWEITE AND H. BERCHEM, *Zement*, Vol. 27, No. 52, Dec. 29, 1938, pp. 815-817.
Reviewed by L. T. BROWNMILLER

In measuring the shrinkage of cements the specimens are sometimes stored over sulphuric acid in order to dry them. The acid gradually absorbs water so that its drying effect changes; abrupt changes in the humidity of the cabinet are recorded when the acid is renewed. The authors propose the substitution of saturated potassium carbonate solution for the acid in order to minimize the changes in relative humidity of the storage cabinet. Some data are given which compare the shrinkage of cements over both types of solutions.

The experimental basis of specifications for highway concrete aggregates

A. KLEINLOGEL, *Beton und Eisen*, Vol. 37, No. 22 (Nov. 20, 1938) p. 360 Reviewed by A. U. THEUER

The author stresses the importance of using properly selected aggregates in concrete structures that are expected to render service in terms of centuries. Aggregates obtained from different sources are stated to vary greatly in physical properties and in any study of concrete durability the aggregates as well as the cementitious material must be given consideration. The author then points to an extensive literature supplying the necessary knowledge regarding the durability and suitability of different types of aggregates. (This knowledge he protests is all too seldom applied in the field). A summary of the data referred to is collected in such a way as to develop the effect of type of aggregate on the compressive, tensile bending, and impact strength, wear, permeability, volume changes, specific gravity, and elasticity of the resulting concrete.

Fast work on reinforced concrete apartments

J. DISTASIO, *Engineering News Record*, Vol. 121, No. 24, Dec. 15, 1938, pp. 759-762.
Reviewed by S. J. CHAMBERLIN

Two-way slab and rigid frame concrete design featured in the building of a large apartment building at Newark, New Jersey. Beams were placed only at the partition walls, giving a low story height of 8 ft. 6½ in. Plywood forms speeded up the work, and in the case of the floors left a ceiling surface easily prepared for direct painting. Floor panels all varied in length but the slabs were readily designed by the A. C. I. code. Maximum bending moments were determined by considering the slabs as continuous. Beams and columns were treated as rigid frames using the fixed point method. No expansion joints were provided, but additional steel was added at the top floor of the columns near the end of the building to take care of temperature stresses. High-early strength cement specified to permit form removal at 3 days, was replaced by standard by redesigning the mix.

Configuration problems in road construction

HANS LORENZ, *Forschungsarbeiten aus dem Strassenwesen*, V. 14, pp. 1-100, 1938.
Reviewed by F. L. EHASZ

This article, consisting largely of sketches, portrays the manner of solving problems in the design of the German parkway systems. It is common knowledge to American highway engineers that national safety councils point to the satisfactory measures taken by the Germans in making travel on highways safe and comfortable. The general layouts, gradients, curves and intersections are so designed as to give the least sensations to the traveling people. Emphasis is placed on the fact that in attacking these configuration problems it is no longer enough to think of simply the plan, longitudinal and tranverse sections; present-day design requires clear visualization in space. Several sketches give illustrations of the following considerations: (1) landscaping; (2) valley crossings; (3) excavations; (4) dividing strips; (5) avenues and alleys; (6) resting places; (7) clover-leaf and squid types of intersection; (8) grade eliminations; (9) parkway lighting; and (10) dwelling places.

Concrete pavements

Revetements Betonnes, Paris, XII (February, June, September and December, 1938)
Reviewed by P. H. BATES

These most excellently printed and illustrated booklets were issued at intervals (February, June, September, and December, 1938) by the Societe Le Soliditit Francais, at 55 Rue de Lyon, Paris. They are intended to cover concrete pavement

construction for highways, airplane runways, river revetments, etc. In the four numbers referred to the following subjects are covered: Improvement of existing highways, urban highways, choice of foundation materials, control of quality of concrete by means of density, airplane runways, and roads in the North African French ports. The December issue is devoted entirely to a digest of the Italian specifications for concrete highways.

The subject matter under these topics is presented in a semi-technical manner and does not go into very great detail. However, they are of much interest, particularly the digest of the Italian specifications, and in this respect, we are promised a digest of the concrete highway specifications of the leading European countries later. The outstanding features, however, of these brochures are the exquisite printing and the illustrations, which it would be difficult to improve upon.

The underpinning of finished structures by made-in-place piles

E. Titze, *Beton und Eisen*, Vol. 37, No. 22 (Nov. 20, 1938), p. 353. Reviewed by A. U. Theuer

The author touches briefly on the subject of foundation failures and then describes in detail applications of made-in-place piles for underpinning operations. Two principal causes of foundation failures are listed: faulty design due to an incorrect analysis or adoption of excessive pressures, and a change in the original foundation condition due to changes unforeseen at the time of construction. A third cause, arising from vibrations, is stated to occur but very rarely. Non-unifrom settlement of a foundation is stated to be the cause of most difficulty in practice. Methods of recognizing corner, midsection or edge settlement from the external appearance of cracks are described. Successive steps in determining the cause as well as the extent of a settlement are given in detail beginning with simple prospecting trenches and leading up through extensive analysis accompanied by laboratory studies.

The second part of the article describing the application of the Lorenz made-in-place pile (characterized by an enlarged bulb of concrete under the toe) does not lend itself well to a brief review. Actual operations carried out on various types of structures are illustrated and described in detail.

German concrete convention reports

Deutscher Beton-Verein, 41. Hauptversammlung, pp. 1-600, March 8-10, 1938
Reviewed by F. L. Ehase

This account of the sessions of the forty-first principal meeting of the German Concrete Institute contains reports of recently prosecuted research and construction, with some historical presentations. The large list of participants given in the first sixty pages or so indicated the great interest in these programs. Some of the material had been promulgated in some form or another in previous technical publications and has been reviewed in recent issues of this Journal. Included among the subjects of discourse were: (1) master builders of the German people; (2) building materials and construction methods in the change of time; (3) Danube bridges of the federal highway at Leipheim; (4) Teufelstal bridge of the federal highway Gera-Jena; (5) reinforced concrete structures of the Central Airdrome, Berlin-Tempelhof; (6) installations on the building yards for the Congress Hall (Nurnberg) and the Saaletal dam (Hohenwarthe); (7) natural-stone bridges of the federal highways; (8) hypotheses and principles for the shrinkage and flow fo concrete; (9) reinforced concrete trusses; (10) surface treatment of concrete and reinforced concrete structures; (11) new reinforced concrete silos; and (12) concrete in fortifications and its behavior against the action of shells.

New reinforced concrete cantilever bridge crossing the River Marne at Bry, France

M. PEYRONNET, *LeGenie Civil*, Vol. CXIII, No. 22, Nov. 26th, 1938, pp .441-445

Reviewed by R. L. BERTIN

This bridge, described in great detail and fully illustrated, is claimed to be the largest reinforced concrete cantilever bridge of its type. In general, the bridge is a single span of 70.75 meters consisting of two cantilever sections each 23.375 meters long supporting a freely supported mid span of 22. m. The cantilevers are counterbalanced with earth filled short spans at each end providing a passage 8.60 meters wide on each shore.

Transversally, the bridge consists of eight frames, spaced 2.16 meters, braced at intervals with diaphragms, carrying arched slabs on top supporting a 10 meter roadway and two 4. m. sidewalks. The cantilever frames are joined at their soffits with an apron extending 18.50 meters from the support which is heavily reinforced and acts as the compression flange of the cantilevers.

The appearance of the bridge is that of an arched structure with a parabolic intrados, having a rise of 3.07 meters, the thickness of the support being 4.24 meters and at the center 1.96 meters.

It is claimed that this type of design proved to be more economical than an arched bridge of equal span.

The effect of dust in stone sand on the properties of mortars

A. T. GOLDBECK, *The Crushed Stone Journal*, Nov.-Dec., 1938, Vol. XIII, No. 6.

Reviewed by J. E. GRAY

This paper is a presentation of data on the effect of dust (defined as minus 100 mesh material) on the water-cement ratio, water-gain, compressive strength, volume change, absorption, and resistance to freezing and thawing of mortars similar to that used in 1:2:3½ concrete having a 2-in. slump. Six different graduations of limestone sand were used in which the dust content varied from 4 to 24 per cent in increments of 4 per cent. The specimens, 2-in cubes and 2x2x14-in. beams, were cured one day in moist air and 27 days in water prior to testing. The results indicate that there was no well defined influence of dust on the water-gain; that the difference in settlement of the fresh mortar was negligible; that the water-cement ratio increased from 0.68 with 4 per cent dust to 0.72 with 24 per cent dust, that the slight increase in w/c was reflected in somewhat decreased strengths; that there is a well defined trend toward higher absorption values as the dust content was increased ranging from 7.6 per cent absorption for 4 per cent dust to 10.2 per cent for 24 per cent dust; that the resistance to freezing and thawing was decidedly lowered with high percentages of dust; and that there was no practical difference in the shrinkage caused by drying in air for one year. Although the tests were made on mortars, the conclusion was that concrete would be similarly affected and for concrete containing 1:2 mortar it seems safe to have 8 to 10 per cent stone dust passing the No. 100 mesh sieve.

Traffic lines cast in color

Contractors and Engineers Monthly, Jan., 1939, p. 32.Highway Research Abstracts, Feb., 1939.

HIGHWAY RESEARCH ABSTRACTS

In the paving of the Galveston Island approach to the new causeway connecting with the city of Galveston, Texas, an interesting feature was the novel manner in which the contractor marked the center line, which consists of a 6-in. yellow stripe between two 6-in. black stripes, incorporating the coloring material in the concrete pavement.

The traffic stripe was formed by working magnetic oxide in yellow and black into the finished concrete to a depth of ¼-in. and hand floating to refinish the stripe. The result of this operation is an attractive and straight stripe, with expectations that the two-color scheme will be a valuable guide to the heavy traffic volumes traveling this thoroughfare.

In order to provide a smooth line for all edges, the contractor developed a hand-propelled machine with an 18-in. hand wheel and a chain drive to the rear axle. All four wheels of this machine were flanged to keep the apparatus in perfect line on the steel road forms. A three-compartment trough 7-ft. long was built in the center of the machine and parallel to the pavement edge. These compartments, composed of structural steel and sheet metal, could be raised by means of a ratchet to permit the operator to move the machine forward. Likewise they could be lowered to the pavement to permit the spreading of the coloring materials and floating them into the surface. The yellow was placed in the center compartment which is directly over the exact center of the pavement and the black placed in each of the side compartments.

Thick shelled reinforced concrete pipes and casings

W. Olszak, *Beton und Eisen*, Vol. 38, No. 2, p. 29 (Jan. 20, 1939) Reviewed by A. U. Theuer

The author comments on the fact that the stress distribution in the walls of heavy pipes subjected to either internal or external pressures is not uniform. In the case of isotropic materials and with internal pressures the tangential stress follows a hyperbolic distribution with a maximum around the inner surface. The outer surface is thus never fully stressed and the material is not utilized up to the point of its full strength. The thicker the pipe becomes the more uneconomic is the resulting structure. In the case of reinforced concrete (anisotropic) an even more uneconomical design results when current practice is followed. The author points out that in the case of anisotropic as against isotropic material, the plain stress and plain deformation conditions are not identical. Further, an ideal solution of this problem would be one requiring the complete utilization of the strength of the material used. The author then points out the principal, that in any statically indeterminate system the weaker members of a structure tend to unload into the stronger members, affording, in this instance, a means for attaining an ideal solution for this problem of thick-walled cylinders. This is accomplished by systematically concentrating the steel around the outer edges of the pipe. Greater elasticity is thereby attained for the zones of the material at greater distances from the center and what the author refers to as a polar or orthotropic material is in effect attained.

He then proceeds to explain the basic principles of his analysis for a specific case. The more involved elastic equations are avoided in this analysis and the reader is referred elsewhere for a rigid solution of the problem.

Static as well as economic advantages are claimed for this design. In resisting stresses the ring will act more nearly as a unit. The high stress concentration around the inner surface is eliminated and the tendency for the development and spread of cracks is reduced. On the economic side for the given case reported, a conventional design required 130 percent more concrete and 12 percent more steel than the design proposed by the author.

A study of the effect of shape of particle on the mortar-making properties of stone sand

A. T. Goldbeck, *The Crushed Stone Journal*, Mar.-Apr., 1938, Vol. XIII, No. 2.
Reviewed by J. E. Gray

Stone sands were prepared in the laboratory with three different shapes of particle and their effect on mortars was studied. In making the three stone sands, the first

was prepared by reducing 1½-2 in. stone to —⅜ in. in a gyratory crusher, the second by passing No. 4 to 50 mesh material from the gyratory through a rotary grinder, and the third by placing a mixture of the former two in a concrete mixer with a charge of steel slugs and rounding off the particles. The sands produced were essentially slabby and angular, cubical, and rounded, respectively. The shape of particle was measured by determining the voids, loose, in the four upper sizes of the sand in definite manner; thus, the No. 8-No. 16 size gave the following percentage of voids; crushed 57.9, ground 51.9, rounded 46.4, and a natural sand 46.0.

In making the mortar tests, the proportions were 1:2 by weight and a consistency corresponding to that of a mortar used in 1:2:3½ concrete having a 2-in. slump. Five gradations were used for each type of sand. The fineness modulus varied from 2.92 to 2.54 and the minus 100 mesh material varied from 3 to 15 per cent. The crushing strength, absorption, and freezing and thawing tests were made when the specimens had been cured for one day in moist air and 27 days in water. The water-cement ratios were the lowest in the mortar with the rounded sand and highest in the moratr with the crushed sand. Consistent with these w/c's, the crushing strength, absorption, and resistance to freezing and thawing was highest with the rounded sand and lowest with the crushed sand. The results indicate that for a given consistency the finer the sand the higher the water content, and the more angular the sand the higher the water content.

In conclusion, it appears that flat and needle-like particles are objectionable; however, prolonged grinding is not necessary, but a marked improvement occurs when cubicle particles are produced.

Some remarks on the present status of the testing of highway cements

OTTO GRAF, *Zement*, Vol. 28, No. 1, Jan. 5, 1939, pp. 1-4. Reviewed by L. T. BROWNMILLER

In 1934 a new testing procedure was necessitated for the German Reich highway cements because of the additional specifications imposed. In addition to the usual tests it became necessary to follow the flexural and compressive strengths under field conditions; and a new test for measuring the shrinkage of cements was introduced.

The first requirement for the new highway cements is a high flexural strength. At the same time a high compressive strength is desirable. Furthermore the strengths, particularly the flexural strength, should be constant under exposure to the weather.

In 1935 a minimum flexural strength of 55 to 60 kg/cm$_2$ was practical at 28 days; today it is possible to manufacture cements with a minimum of 75 kg/cm$_2$. The author is not particularly informative as to how those increases in strength were obtained but merely intimates that the improvements were brought about by changes in the composition of the cements or by changes in the process of manufacture or by changes in both.

The measurement of the shrinkage of the cements as carried out to the present, has provided useful information, but probably the test will have to be changed in order to duplicate the effects of shrinkage in actual service. The small mortar specimens which were used for test purposes showed considerable differences in the amount of contraction; however in concrete the variations between the different cements were negligible.

Much more important was the effect of drying on the flexural strength of the concrete. In some cases the flexural strength decreased by 50 per cent on drying for 28 days. The solution of how to avoid such changes is a problem still to be solved.

The objection has often been raised that the test specimens are subjected to very severe conditions which are not duplicated in concrete in actual service. The author therefore offers some data on large concrete beams which show that the changes in flexural strength are essentially the same regardless of whether the beam dries out from one side only or from all four sides simultaneously.

Fluates and their limitations as effective surface treatments

C. R. PLATZMANN, *Tonindustrie Zeitung*, Vol. 63, No. 7, p. 71 (Jan. 23, 1939)

Reviewed by A. U. THEUER

The wide field of application of fluate treatments as protective coatings for masonry surfaces is commented upon. At the same time the author adds a word of warning that fluates are not a cure-all for all the difficulties met in practice. First proposed as a treatment for exposed natural stone surfaces about 1880, the use of fluates has since been widened to include treatments of concrete and mortar.

The action of a fluate applied to the surface of concrete is explained as follows: The hardening of concretes is accompanied by a release of lime in the form of hydrate in varying amounts of from 7 to 17 per cent. In the presence of air the lime combines with carbonic acid resulting in a compound of minor importance in the hardening (hydraulic) process and one that is undesirable for two reasons; (1) the resulting compound is very soft, giving rise to dusting in the case of floors; (2) under the influence of acids (even very weak ones) the compound gives rise to efflorescence. In the presence of a fluate such as magnesium fluoride the following chemical transformation is said to follow: $3MgSiF_6 + 6CaCO_3 \rightarrow 6CaF_2 + 3MgF_2 + 3SiO_2 + 6CO_2$. The mineral combination resulting from this transformation is both harder and more stable than the original one.

As an example of an effective treatment for concrete floors the following data are presented. (The treated specimen was impregnated with a magnesium fluoride solution at the age of 14 days and an abrasion test was carried out by the Boehm method, 200 revolutions.)

	Specimens	
	Plain	Treated
Original weight, grams	750	770
Weight after abrasion test	429	742
Weight loss, percent	41	3.5

As an example of effective waterproofing the following test of two mortar cubes is presented:

	Permeability	
	Plain	Treated
Water leakage in c.c.		
after 1 minute	1.132	0.135
after 30 minutes	0.186	0.074
after 60 minutes	0.174	0.046

The author stresses the fact that fluate treatments do not serve as protection against acid attack. (A relative or percentage increase in acid resistance may be attained.)

The author then discusses methods of treatment, stressing the importance of choosing a proper concentration which must be varied with the structure of the

material treated. Successful application is stated to require experience as well as a study of the material to be treated.

A discussion of the use of fluate for the treatment of natural stone and timber follows.

Concreting in winter

E. BORNEMANN, *Der Bautenschutz*, Vol. 9, No. 11, p. 11 (Nov. 5, 1938) Reviewed by A. U. THEUER

The author presents a general discussion and summary of conclusions regarding the practice of winter concreting. He points out that, with concrete strength, a function of the curing temperature as well as the elapsed time after pouring, more than subfreezing temperatures is involved. With temperatures below 0°C., however, difficulties increase and special precautions become essential. The exact strength to be attained by a concrete before permitting the possibility of ice formations in the mass is considered by the author to be a moot question. German specifications require that the temperature of concrete shall not drop below +2°C. during the first 72 hours after pouring. Author would replace this recommendation by one requiring that a concrete at the age of 72 hours, under any conditions of curing, must be equal to the strength of a concrete at the age of 24 hours when cured under normal conditions. Methods to obtain this desired result, wherever possible, without the use of artificial heat, are then enumerated. Preheating of the mixing water as a method of raising the temperature of a mix before pouring is considered the most satisfactory of elementary methods. Due to the small amount of mixing water involved, however, this method is limited in its application. Author here recommends that cement never be introduced to a mix (water + aggregate) whose temperature is higher than 35°C. due to the possibility of flash set. Preheating in a mixer due to reduced plant capacity is another method that can be used only within relatively narrow limits. When temperatures fall very far below 0° C. preheating of the aggregates is required. The advantages of steam as against hot gases are enumerated, and some physical data concerning boiler capacities and efficiencies obtained under specific conditions are given.

In order to prevent a too rapid cooling of a fresh mix after placement, the use of quick setting cements are recommended. Not all high early-strength cements, it is stated, are superior in this respect to normal portlands. Data showing the times of set of a number of cements cured at 0°C. (air temperature) illustrate the large differences for different brands of cements. Richness of mix, size of body, insulation, etc., are discussed in order. Cautions regarding the use of integral chemicals are stated. Author would require preconstruction tests carried out under actual field conditions before permitting their use in important structures.

The application of heat after placement leads to the following recommendations: (1) All structural elements must be heated uniformly and sufficiently throughout. (2) Outer members require the direct application of heat to all sides. (3) It is not sufficient to heat the underside of slabs only. Openings for the passage of heat should be provided in all slabs. (4) The space to be heated must be well insulated. (5) The heating plant capacity must be such that concrete temperatures above 15°C. can be maintained. (The additional time of heating required wherever a lower temperature is maintained is stated to offset any possible economices due to reduced plant capacity); (6) With open fires or ovens, fire prevention apparatus must be provided. (7) Freshly poured sections must be protected against direct radiation.

The ideal is stated to be the complete enclosure of the entire construction area. In considering costs, the increased efficiency of labor must be given due consideration.

Merely as an aid in formulating an answer to the question—is it more economical to pay the increased costs involved in winter construction than to await the arrival of suitable weather?—the following is offered:

(1) Between 0 and +10° C. precautionary measures involve only negligible costs compared to total costs and delays in commencing construction would seldom be justified; (2) In climates where low temperatures are continuous and usual, winter concreting is more often justified than in the case of warmer climates; (3) Reasons for going ahead with construction during freezing weather should be real; (4) Wherever freezing weather may be expected during concrete construction, costs must be provided for in the plans; (5) Type of structure and the required quality of workmanship must be considered; (6) A rough idea of the increase in costs involved whenever artificial heat is used are given as from 1 to 5% for large structures and from 4 to 8% for subsurface structures; (7) Reasonable estimates of the costs involved are almost always possible. Labor productivity enters as an important factor; (8) Costs for protective elements are never excessive, but owners must be brought to a realization that only with pre-construction planning can the ensuing result ever be wholly satisfactory.

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Application of the Results of Research to the Structural Design of Concrete Pavements*

By E. F. Kelley†

SYNOPSIS

The results of research, applicable to the structural design of concrete pavements, are reviewed and their significance indicated. It is shown that the critical stresses in a pavement slab are primarily due to single wheel loads and not to axle loads or gross weight of vehicle; that impact forces considerably in excess of static wheel loads should be used in the design of pavements; and that the stress analyses of Westergaard, with the modifications suggested by the Arlington tests of the Bureau of Public Roads, are suitable for use in pavement design. The simplified methods, developed by Bradbury, for computing stresses due to load and temperature warping, are explained and the necessary tables included.

It is shown that the warping stresses due to differentials of temperature within the slab may be as great as the stresses due to heavy wheel loads and that if transverse cracking due to these stresses is to be avoided, short slabs must be used. It is also shown that, if a pavement is designed for heavy wheel loads in such manner that the combined stresses due to load and temperature are limited to safe values and are reasonably well balanced, the thickened-edge cross section has no particular advantage over the cross section of uniform thickness.

The design of steel reinforcement and its advantages are discussed. The need for transverse and longitudinal joints is emphasized and information regarding their design is given.

During the past 20 years many studies have been made of the various factors which influence the structural performance of concrete pavement slabs and the numerous reports of these investigations are scattered through the technical literature. Most of these reports, of

*Digest of paper presented at the 35th Annual Convention of the American Concrete Institute, New York, March 1-3, 1939. The complete paper is scheduled for publication by the Bureau of Public Roads.

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necessity, are highly technical and the mass of data presented and the detailed descriptions that are included, both as a matter of record and in order that the reader might have confidence in the validity of the results, frequently tend to obscure the value and importance of the conclusions. Also, in many cases, individual reports cover but a single phase of a given subject and are useful only when considered in connection with the available reports dealing with the remaining phases of the same subject. The net result of this situation is that many facts that have been well established by research are little appreciated and too frequently are given scant consideration in the practical design of pavements. It is the purpose of this paper to bring together under one head and make available for the practical use of the designing engineer the important facts that have been developed thus far in research work relating to the structural design of concrete pavements.

WHEEL LOADS AND IMPACT

Neglecting the unpredictable forces caused by localized differential heaving or subsidence of the subgrade soil, the external forces that create stress in the pavement slab are produced by vehicles. Naturally, the heavier vehicles are the more important.

One of the earlier investigations (1)[1] developed the important fact that for heavy vehicles of the usual type, that is, four or six-wheel trucks or trailers, the critical stress developed in a concrete pavement, when the axle spacing is in excess of about three feet, is primarily a function of the wheel load and not a function of the gross load on the vehicle or the axle spacing. By means of his theoretical analysis, Westergaard (2) subsequently arrived at the same conclusion and this has been confirmed by later tests (3). This finding, which permits us to confine our attention to wheel loads rather than gross loads, greatly simplifies a problem which is already sufficiently complicated.

The magnitude of the vertical force exerted on a pavement by the wheel of a moving vehicle may be considered to be the sum of the static weight of the loaded wheel and the additional impact or dynamic force created by the movement of the wheel over the irregularities that exist in the pavement surface. The researches of the Bureau of Public Roads have demonstrated conclusively that the impact reaction of a moving wheel is sufficiently in excess of the static wheel load to make it an important factor in pavement design.

On the basis of these researches impact factors suitable for use in the design of pavements are given in Table 1 for a range of static

[1]Figures in parentheses refer to the bibliography.

TABLE 1—IMPACT FACTORS AND TOTAL IMPACT ROAD REACTIONS
Speed—50 miles per hour
Frequency—100 per mile
Condition of pavement surface—reasonably smooth

Static Wheel Load	Dual High-Pressure Tires		Dual Balloon Tires	
	Impact Factor	Total Impact Reaction	Impact Factor	Total Impact Reaction
Pounds		Pounds		Pounds
4,000	2.05	8,200	1.70	6,800
5,000	1.80	9,000	1.54	7,700
6,000	1.67	10,000	1.43	8,600
7,000	1.56	10,900	1.37	9,600
8,000	1.48	11,800	1.31	10,500
9,000	1.41	12,700	1.27	11,400
10,000	1.36	13,600	1.24	12,400

loads on wheels equipped with dual high-pressure and with dual balloon tires. It will be observed that the impact reaction, or total impact force, increases as the static wheel load increases but that the impact factor (ratio of total impact force to static wheel load) decreases as the wheel load increases. It will also be noted that, for a given wheel load, the impact reaction for balloon tires is less than for high-pressure tires.

FATIGUE LIMIT OF CONCRETE

Concrete, like other structural materials, will fail under repeated loads at unit stresses which are much less than the ultimate strength as determined by the stress at failure produced by one application of static load. The stress at which failure takes place under a very large number of loadings is known as the fatigue limit or the endurance limit and, in the case of concrete, it is expressed as a percentage of the ultimate strength.

On the basis of available data it has become rather general practice to assume about 50 per cent of the ultimate flexural strength as a safe value of the working stress for use in designing pavements to resist wheel loads. However, the fatigue limit of the order of 50 per cent of the ultimate flexural strength of the concrete has been established by tests in which the load applications were repeated at relatively short time intervals. In contrast to this, under normal conditions of traffic the heavy wheel loads that produce maximum stress are applied to the pavement slab at relatively long time intervals. It is by no means certain that the fatigue limit might not be considerably different, and possibly higher, for stresses applied at time intervals corresponding to those which occur between successive applications of heavy wheel loads to a pavement under traffic.

It is a well-known fact that stresses above the fatigue limit cause progressive inelastic deformation and final failure. However, the

relation between intensity of stress above the fatigue limit and the number of repetitions of this stress that will cause failure is not well established even for rapid repetitions of stress. For less frequent repetitions nothing is known concerning it.

On the majority of highways the heavier trucks constitute a small percentage of the total traffic and therefore the occurrence of maximum load stresses is relatively infrequent. It appears therefore that the present practice of assuming the design stress to be approximately 50 per cent of the ultimate strength of the concrete is a conservative one insofar as the stresses due to maximum wheel loads are concerned. In view of the possibility that the fatigue limit for these infrequent repetitions of stress may be higher than is indicated by available data, this may introduce some factor of safety of unknown magnitude.

STATIC STRESS VERSUS IMPACT STRESS

At the present time we know very little about the relation between the stresses produced in concrete pavement slabs by static and impact forces of the same magnitude. There is some evidence, by no means conclusive, indicating that under certain conditions the stresses due to impact forces may be less than those produced by static forces.

However, even if significant differences are eventually found to exist between static and impact stresses in a pavement slab, we have no means for evaluating them at this time and therefore must make the assumption that impact forces create the same stresses as static forces of the same magnitude. It appears that this is a safe practice and one which may introduce some factor of safety that at present is unknown.

MATHEMATICAL ANALYSIS OF STRESS

In 1919 Goldbeck (16) suggested approximate formulas for computing the stresses in concrete pavement slabs under certain assumed conditions of loading and subgrade support. Among these approximate formulas is one which has since become generally known as the "corner formula." This may be expressed in the form

$$\sigma_c = \frac{3P}{h^2} \qquad (1)$$

where σ_c = maximum tensile stress, in pounds per square inch, in a diagonal direction in the top of the slab near a rectangular corner

P = load, in pounds, applied at a point at the corner

h = depth of slab in inches

This simple formula is derived on the assumption that the load is applied at a point at the extreme corner of the slab; that the corner receives no support from the subgrade and acts as a simple cantilever; and that the fiber stresses in the slab are uniform on any section at right angles to a line bisecting the corner angle.

Some years later, in the analysis of the data from the Bates Road tests (17), it was found that there was a reasonably good agreement between the wheel loads that caused corner failure and loads computed by the corner formula. However, it is now quite definitely known that the corner formula gives stresses considerably higher than the actual stresses in pavement slabs, even under extreme conditions of warping. The agreement between computed loads and measured loads in the Bates Road report may be explained by the fact that the latter were static wheel loads while the loads which actually caused corner failures were the impact reactions due to these wheel loads. In view of the fact that the truck wheels were equipped with solid rubber tires, the impact loads were undoubtedly considerably higher than the static wheel loads.

In 1925 the analysis of Westergaard (2) made available for the first time a logical and scientific basis for evaluating the stresses in concrete pavements. This analysis concerns itself with the determination of maximum stresses in slabs of uniform thickness resulting from the following three conditions of loading:

1. Load applied close to the rectangular corner of a large slab.
2. Load applied in the interior of a large slab at a considerable distance from the edges.
3. Load applied at the edge of the slab at a considerable distance from any corner.

For the three positions of load, the analysis results in equations which may be expressed as follows:

$$\sigma_c = \frac{3P}{h^2}\left[1 - \left(\frac{12\,(1-\mu^2)k}{Eh^3}\right)^{0.15}(a\sqrt{2})^{0.6}\right] \quad (2)$$

$$\sigma_i = 0.275\,(1+\mu)\,\frac{P}{h^2}\log_{10}\left(\frac{Eh^3}{Kd^4}\right) \quad (3)$$

$$\sigma_e = 0.529\,(1+0.54\mu)\,\frac{P}{h^2}\left[\log_{10}\left(\frac{Eh^3}{kb^4}\right) - 0.71\right] \quad (4)$$

in which

P = load, in pounds

σ_c = maximum tensile stress in pounds per square inch at the top of the slab, in a direction parallel to the bisector of the corner angle, due to a load P at the corner

σ_i = maximum tensile stress in pounds per square inch at the bottom of the slab directly under the load P, when P is at a point in the interior of the slab at a considerable distance from the edges

σ_e = maximum tensile stress in pounds per square inch at the bottom of the slab directly under the load P at the edge, and in a direction parallel to the edge

h = thickness of the concrete slab, in inches

μ = Poisson's ratio for concrete

E = modulus of elasticity of the concrete, in pounds per square inch

k = subgrade modulus, in pounds per cubic inch

a = radius of area of load contact, in inches. The area is circular in the case of corner and interior loads and semicircular for edge loads

b = radius of equivalent distribution of pressure

$$b = \sqrt{1.6a^2 + h^2} - 0.675h \text{ when } a < 1.724h \quad \ldots\ldots\ldots\ldots (5)$$

$b = a$ when a $> 1.724h$

VALUE OF POISSON'S RATIO

A review of available data indicates that the general practice, first suggested by Westergaard, of assuming for the purpose of pavement design that Poisson's ratio is equal to 0.15, is an entirely reasonable one and that value will be used throughout this paper.

In addition to the quantities which appear directly in the three stress equations, we have the radius of relative stiffness, l, which is defined by the equation

$$l = \sqrt[4]{\frac{Eh^3}{12(1-\mu^2)k}} \quad \ldots\ldots\ldots\ldots (6)$$

Westergaard has expressed Equation 2 in terms of l, as follows:

Corner loading

$$\sigma_c = \frac{3P}{h^2}\left[1 - \left(\frac{a\sqrt{2}}{l}\right)^{0.6}\right] \quad \ldots\ldots\ldots\ldots (7)$$

and Bradbury (9) has shown that, when $\mu = 0.15$, Equations 3 and 4 may be expressed in the form:

Interior loading

$$\sigma_i = 0.31625\,\frac{P}{h^2}\left[4\,\log_{10}\left(\frac{l}{b}\right) + 1.0693\right] \quad \ldots\ldots\ldots\ldots (8)$$

Edge loading

$$\sigma_e = 0.57185 \frac{P}{h^2}\left[4 \log_{10}\left(\frac{l}{b}\right) + 0.3593\right] \quad \ldots\ldots\ldots\ldots (9)$$

MODIFIED EQUATIONS FOR CORNER LOADING

If, in Equation 2, for corner loading, the radius of contact area, a, is assumed equal to zero then the influence of the subgrade modulus, k, and the modulus of elasticity, E, are eliminated and the equation reduces to the corner formula

$$\sigma_c = \frac{3P}{h^2} \quad \ldots\ldots\ldots\ldots (1)$$

In a somewhat limited but carefully conducted series of tests on large slabs under laboratory conditions Spangler and Lightburn (22, 23) observed corner stresses appreciably greater than those computed by the Westergaard equation.

As a result of these observations Bradbury (9) has suggested the modified equation

$$\sigma_c = \frac{3P}{h^2}\left[1 - \left(\frac{a}{l}\right)^{0.6}\right] \quad \ldots\ldots\ldots\ldots (10)$$

In the Arlington tests (15e)[2], in which the slabs were exposed to normal weather conditions, it has been found that in the daytime, when the corner is warped downward and has contact with the subgrade, there is very good agreement between observed stresses and those computed by the Westergaard Equation 7. However, at night, when the corner is warped upward, the observed stresses, while lower than those given by the corner formula, are much higher than those computed either by the Westergaard equation or by Bradbury's Equation 10.

In the Arlington tests it has been found that when the slab is warped upward the observed stresses are in good agreement with stresses computed by the equation

$$\sigma_c = \frac{3P}{h^2}\left[1 - \left(\frac{a\sqrt{2}}{l}\right)^{1.2}\right] \quad \ldots\ldots\ldots\ldots (7a)$$

It will be observed that this equation has the same general form as the Westergaard Equation 7 and Bradbury's Equation 10. However, it is purely empirical and has no theoretical background. Its only virtue is its algebraic simplicity and the fact that it gives results that are in reasonably good agreement with a considerable number

[2]The term "Arlington tests" will be used to designate the investigation of concrete pavement design made by the Bureau of Public Roads at the Arlington Experiment Farm and described in reports listed in the bibliography (15).

of tests on pavement slabs exposed to normal fluctuations of temperature and moisture. Its use is suggested pending the time that more exact information may be available.

A comparison of the results given by Equations 1, 7, 7a and 10 is shown in Fig. 1. For the range of conditions assumed, the corner stresses computed by Westergaard's Equation 7 are exceeded by those computed by Bradbury's Equation 10 by 7 to 20 per cent, by those computed by Equation 7a by 27 to 51 per cent, and by those computed by the corner formula, Equation 1, by 38 to 104 per cent.

MODIFIED EQUATIONS FOR INTERIOR LOADING

Early in the Arlington tests it was found that the observed stresses due to loads in the interior of the slab were not as great as those computed by Equation 3 and as a result Westergaard modified his original analysis (24). The modified equation for stress due to interior loading is

$$\sigma_i = 0.275(1 + \mu)\frac{P}{h^2}\left[\log_{10}\left(\frac{Eh^3}{kb^4}\right) - 54.54\left(\frac{l}{L}\right)^2 Z\right] \quad \ldots\ldots (3a)$$

in which

L = maximum value of the radius of the circular area, with center at the point of load application, within which a redistribution of subgrade reactions is made

Z = ratio of reduction of the maximum deflection

The reduction of interior stresses, as expressed by Equation 3a, is dependent on the characteristics of the subgrade and the slab and the complex reaction between them and these variables may be quite different at different locations. Therefore it appears that Equation 3a is not suitable for any general use and that it will be preferable to continue to use the results given by the original Westergaard analysis, Equation 8.

MODIFIED EQUATION FOR EDGE LOADING

In the Arlington tests it has been found that for what may be considered as average values of a, the radius of contact area, there is good agreement between observed edge stresses and those computed by Westergaard's Equation 9 when the slab is in an unwarped condition.

When the edges of the slab are warped upward at night the observed load stresses exceed the theoretical stresses, as in the case of corner loading although not to the same extent. It has been found that the observed stresses under the conditions of night-time warping are in reasonably good agreement with the empirical equation

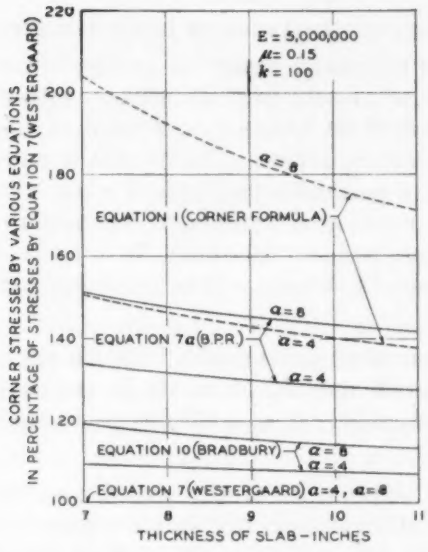

FIG. 1—COMPARISON OF CORNER STRESSES COMPUTED BY VARIOUS EQUATIONS

$$\sigma_e = 0.57185 \frac{P}{h^2}\left[4 \log_{10}\left(\frac{l}{b}\right) + \log b\right] \quad \text{(9a)}$$

SIMPLIFICATION OF STRESS COMPUTATIONS

The equations of Westergaard and the modified equations that have been discussed are simple algebraic expressions but their solution requires a considerable amount of tedious labor. However, Bradbury (9) has suggested a simplified method of computation which reduces the determination of stress by means of these equations to a simple slide-rule operation.

He has pointed out that all the equations have the general form

$$\sigma = \frac{CP}{h^2} \quad \text{(11)}$$

in which C is a quantity that may be termed a stress coefficient. The coefficients C_i and C_e, for interior and edge stresses, respectively, are fixed by the ratio l/b; while the coefficient C_c for corner stresses is fixed by the ratio a/l.

The complete paper contains tables of values of b, l, and the various stress coefficients. Stress computations are greatly simplified by the use of these tables.

EFFECT OF VARIABLES ON COMPUTED STRESSES

For a specific pavement design to be used in a specific location it is not possible at present to predetermine, with any degree of precision, the values to be assigned to several of the variables which appear in the stress equations. Therefore it is necessary, both when the design is for a particular project and when it is a general design to be used on a number of projects, to assign reasonable and rather conservative values to these variables. In order to do this it is necessary to have some knowledge of their relative effects on computed stresses.

A consideration of these variables leads to the conclusion that a value of the subgrade modulus, k, of 100 lb. per cu. in. and a value of the modulus of elasticity, E, of 5,000,000 p.s.i., are reasonable figures for general use.

The radus of contact, a, of the loaded area has an important influence on computed stresses and data are presented in the complete paper showing the value of this radius for a range of wheel loads on high-pressure and balloon tires.

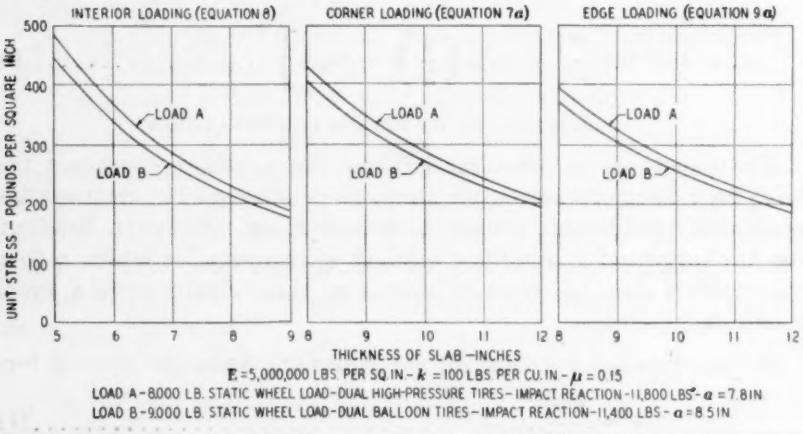

FIG. 9—COMPARISON OF STRESSES DUE TO 8,000 POUND WHEEL LOAD ON HIGH PRESSURE TIRES AND 9,000 POUND WHEEL LOAD ON BALLOON TIRES

VARIATIONS IN THICKNESS OF SLAB, h

The relation between slab thickness and load stresses is shown graphically in Fig. 9 for two loads; one a static load of 8,000 lb. on a wheel equipped with dual high-pressure pneumatic tires and the other a static load of 9,000 lb. on a wheel equipped with dual balloon tires. The impact reactions corresponding to these wheel loads are taken

from Table 1 and the corresponding radii of contact areas from a figure not presented in this digest. For the slab thicknesses ordinarily encountered in practice, the heavier wheel load on balloon tires gives stresses lower than those for the lighter wheel load on high-pressure tires by about 20 p.s.i. Here is justification for the requirement of the Uniform Vehicle Code (28) that the maximum wheel load on high-pressure tires be limited to 8,000 pounds and that on balloon tires to 9,000 lb.

WARPING STRESSES DUE TO TEMPERATURE DIFFERENTIAL

Changes in the temperature of concrete produce corresponding changes in its volume. A rise in temperature causes expansion of the concrete and a drop in temperature causes it to contract.

The temperature of a concrete pavement is constantly changing due to variations in air temperature and during these changes in air temperature, which take place at a relatively rapid rate, the temperature in the slab does not remain constant throughout its depth. During the heat of the day in summer the top of the slab is warmer than the bottom while at night the reverse may be true. This differential in temperature between the two surfaces of the slab cause it to warp or curl and, since free warping is prevented by the weight of the slab, bending stresses are developed.

As early as 1926 Westergaard (29) presented a theoretical analysis of warping stresses due to temperature but their importance has not been generally recognized, possible due to the fact that in his stress computations he assumed a rather low value for the temperature differential. It remained for the Arlington tests (15b) to demonstrate that these warping stresses may be as great as those produced by heavy wheel loads.

VALUE OF TEMPERATURE DIFFERENTIAL

The data developed in the Arlington tests (15b) showed that the maximum temperature differential varies with the depth of the slab, being greater in thick slabs than in thin ones. The maximum positive differential occurs in the daytime and is greater in summer than in winter. The maximum negative differential occurs at night and is much the same in both winter and summer.

From these data Bradbury (9) concluded that, for purposes of design computations, the maximum positive temperature differential might be assumed as 3.0°F. per inch of slab thickness and the maximum negative differential as 1.0°F. per inch of slab thickness. These appear to be reasonable figures for general use but it should be recognized that they are merely average figures and will result in computed

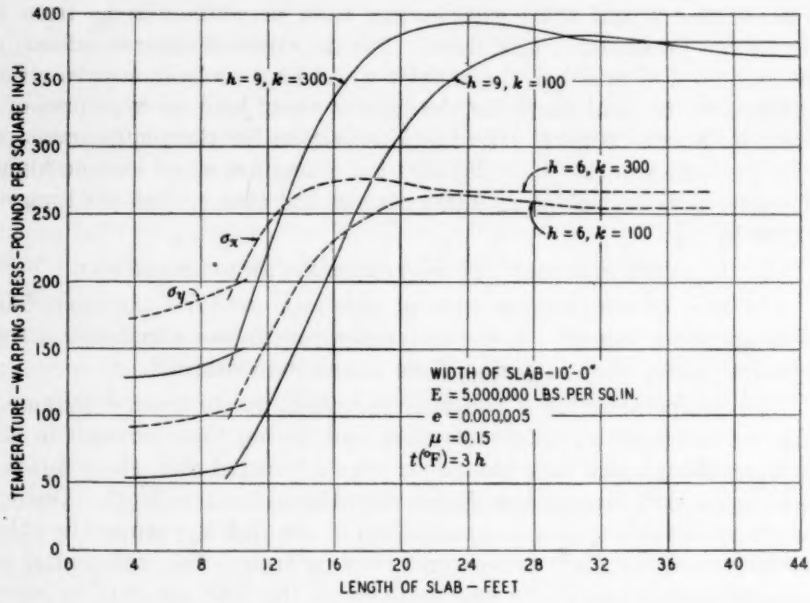
TEMPERATURE – WARPING STRESS – POUNDS PER SQUARE INCH
LENGTH OF SLAB – FEET
h = 9, k = 300
h = 9, k = 100
h = 6, k = 300
h = 6, k = 100
σx
σy
WIDTH OF SLAB – 10'-0"
E = 5,000,000 LBS. PER SQ. IN.
e = 0.000,005
μ = 0.15
t(°F) = 3 h

FIG. 11—TEMPERATURE-WARPING STRESSES—INTERIOR OF SLAB

stresses that may be appreciably lower than the stresses that will occur at times in the pavement. In the Arlington tests values of the positive temperature differential as high as 4.0°F. per inch of slab thickness were observed occasionally.

COMPUTED WARPING STRESSES

The Arlington tests were all made on slabs which varied in dimensions only in depth. Within these limitations the observed warping stresses due to temperature differential were in reasonably good agreement with computed stresses.

Computed temperature warping stresses are shown graphically in Fig. 11 for the interior, and in Fig. 12 for the edge, of slabs 10 ft. wide and of varying length, depths of 6 and 9 in. and values of the subgrade modulus of 100 and 300 lb. per cu. in.

The most striking fact shown by these curves is the magnitude of the maximum temperature-warping stresses, which are of the order of 275 and 375 p.s.i., respectively, for the 6-in. and 9-in. slabs. Other interesting observations that may be made are enumerated as follows:

1. Increases in the length of the slab beyond about 18 ft. in the case of the 6-in. slab, and about 24 ft. in the case of the 9-in. slab, have no

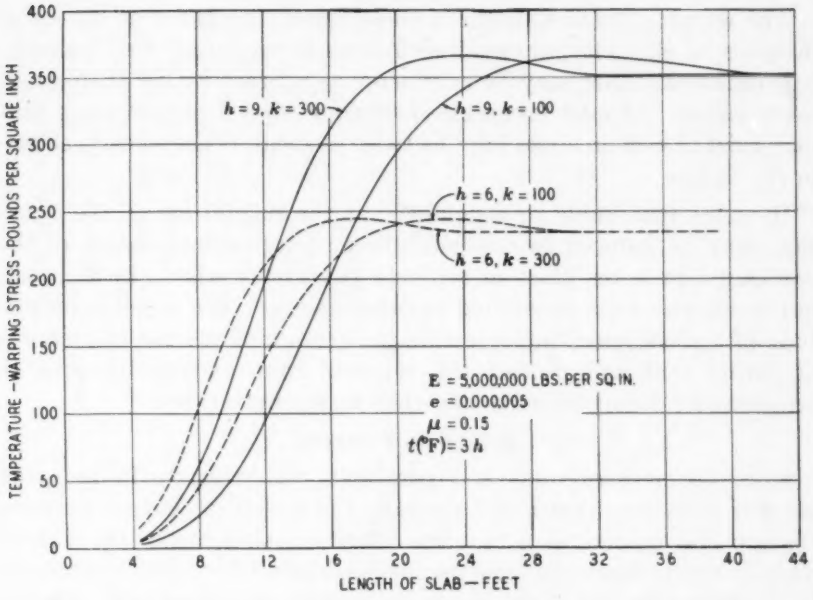

FIG. 12—TEMPERATURE-WARPING STRESSES—EDGE OF SLAB

great influence on maximum edge or interior stresses. Below these limits, decreases in slab length result in rapid reduction in stress.

2. For the longer slabs the maximum stresses in the 9-in. slab exceed those in the 6-in. slab by 40 to 50 per cent. However, for slab lengths less than about 17 feet for $k = 100$ and 13 feet for $k = 300$, the stresses in the 6-in. slab exceed those in the 9-in. slab by as much as 50 p.s.i.

3. Variations in the value of the subgrade modulus have no significant influence on the stresses in long slabs. However, for short slabs increases in the value of the subgrade modulus result in considerable increases in the computed stresses. Fig. 11 and 12 show that the stresses in the 9-in. slab for $k = 300$ may exceed those for $k = 100$ by more than 100 p.s.i.

This effect of subgrade modulus on temperature stresses is the reverse of its effect on stresses due to wheel loads where low values of the modulus give higher stresses than do high values. In the case of combined stresses due to load and temperature warping this reversal of influence tends to compensate somewhat for possible errors in computed stresses due to the assumption of a subgrade modulus different from that which may actually exist.

The warping stresses shown in these figures are based on values of modulus of elasticity, thermal coefficient of expansion and temperature differential that may be considered as average rather than maximum values. Under favorable conditions the warping stresses that may exist at certain times may be twice as great as the stresses shown in the figures.

It seems reasonable to conclude that the magnitude of the stress that may be induced by temperature warping explains much of the cracking that takes place in concrete pavements which, in the past, has frequently been attributed to other causes. The possible magnitude of these stresses indicates the importance of the use of methods of curing that will protect the concrete from extreme changes of temperature during its early life when its strength is low.

MOISTURE WARPING

Since concrete expands and contracts with changes in moisture content it follows that a difference in the moisture content between the top and bottom of a concrete pavement slab causes the slab to warp or curl in much the same manner as does a differential in temperature. When the top of the slab is dryer than the bottom the edges of the slab curl upward and when the moisture differential is in the opposite direction the edges of the slab curl downward.

The strains due to moisture warping that have been measured in connection with the Arlington tests cannot be translated into stress with any certainty. However, the observations made indicate that the curvature caused by moisture is principally an upward warping of the edges caused by moisture loss from the top of the slab during the warm season of the year and that the downward warping which takes place when the moisture in the top of the slab exceeds that in the bottom may be expected to be considerably smaller. Thus, during hot summer days when moisture and temperature differentials are both a maximum, the curvature caused by one is in the opposite direction to that caused by the other and such stress as may be caused by moisture serves to reduce rather than to increase the stress due to temperature warping. Since we cannot evaluate the stresses due to moisture warping it is fortunate that the evidence indicates that they may be disregarded with safety in computing the stresses in pavement slabs. To ignore them appears to add some factor of safety of unknown magnitude and importance.

COMBINED STRESSES

Total combined edge stresses due to load and temperature warping are shown in Fig. 15 for slabs of different thicknesses having lengths

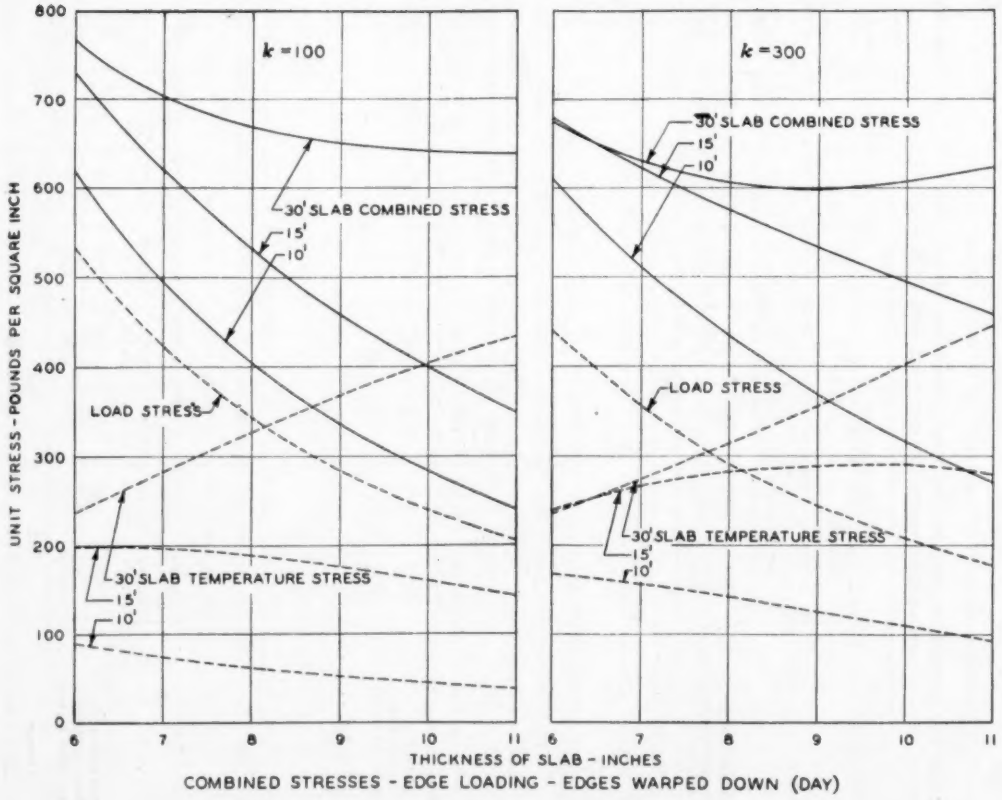

FIG. 15—EFFECT OF SLAB THICKNESS, SUBGRADE MODULUS AND SLAB LENGTH ON COMBINED STRESSES DUE TO LOAD AND TEMPERATURE WARPING IN THE EDGE OF A SLAB 10 FT. WIDE

Load—8,000 pounds static wheel load—dual high-pressure tires. Impact reaction — 11,800 lbs. —a = 7.8 inches. Load stresses by equation 9. E = 5,000,000 pounds per square inch. μ = 0.15. e = 0.000005. t(°F) = 3h (inches)

TABLE 13—COMBINED EDGE AND INTERIOR STRESSES IN SLABS 10 FEET WIDE AND OF UNIFORM THICKNESS

Depth of Slab	Position	Length of Slab					
		30 Feet		15 Feet		10 Feet	
		k = 100	k = 300	k = 100	k = 300	k = 100	k = 300
Inches		Lb. per sq. in.	Lb. per sq. in.	Lb. per sq. in.	Lb. per sq. in.	Lb. per sq. in.	Lb. per sq. in.
9	Interior	570	550	380	480	250	320
	Edge	650	600	460	530	330	370
8.3	Interior	570	550	420	500	290	350
	Edge	660	600	510	560	380	410

TABLE 14—COMBINED STRESSES IN THICKENED-EDGE SLABS AND SLABS OF UNIFORM THICKNESS SLABS 10 FEET WIDE AND 10 FEET LONG

	9-6-9-inch Section				7.1-inch Uniform Thickness			
	Interior		Edge		Interior		Edge	
	k = 100	k = 300	k = 100	k = 300	k = 100	k = 300	k = 100	k = 300
	Lb. per sq. in.	Lb. per sq. in.	Lb. per sq. in.	Lb. per sq. in.	Lb. per sq. in.	Lb. per sq. in.	Lb. per sq. in.	Lb. per sq. in.
Load stress	370	320	430	370	280	250	410	350
Warping stress	110	200	50	130	90	180	70	150
Combined stress	480	520	480	500	370	430	480	500
Average	500		490		400		490	

	10-6.8-10-inch Section				8-inch Uniform Section			
	Interior		Edge		Interior		Edge	
	k = 100	k = 300	k 100 =	k = 300	k = 100	k = 300	k = 100	k = 300
	Lb. per sq. in.	Lb. per sq. in.	Lb. per sq. in.	Lb. per sq. in.	Lb. per sq. in.	Lb. per sq. in.	Lb. per sq. in.	Lb. per sq. in.
Load stress	300	260	370	320	230	200	340	290
Warping stress	90	180	50	110	70	170	60	140
Combined stress	390	440	420	430	300	370	400	430
Average	415		425		335		415	

	11.2-7.8-11.2-inch Section				9-inch Uniform Section			
	Interior		Edge		Interior		Edge	
	k = 100	k = 300	k = 100	k = 300	k = 100	k = 300	k = 100	k = 300
	Lb. per sq. in.	Lb. per sq. in.	Lb. per sq. in.	Lb. per sq. in.	Lb. per sq. in.	Lb. per sq. in.	Lb. per sq. in.	Lb. per sq. in.
Load stress	240	210	310	270	190	170	280	240
Warping stress	70	170	40	90	60	150	50	130
Combined stress	310	380	350	360	250	320	330	370
Average	345		355		285		350	

of 10, 15 and 30 ft. The curves for combined interior stresses are of similar form and are not shown in this digest. The assumed load is an 8,000 lb. wheel on high-pressure tires.

Bearing in mind that the temperature warping stresses shown in this figure may be regarded as average rather than probable maximum values, the following interesting observations may be made with respect to the combined edge stresses and the combined interior stresses in a slab 10 ft. wide.

1. In slabs 30 ft. long an increase in the depth of slab does not affect any marked decrease in the total combined stress. In fact, for $k = 300$, there is a slight increase in the edge stress as the thickness is increased beyond 9 in.

2. Reducing the slab length from 30 to 15 ft. results in some reduction in stress when $k = 100$ but has very little effect when $k = 300$.

3. Reducing the slab length from 30 ft. to 10 ft. results in an appreciable reduction in combined interior and edge stresses.

It may be concluded, for the stress-producing conditions assumed, that:

1. In slabs as long as 30 ft., the depth of slab has very little influence on the magnitude of combined interior and edge stresses.

2. In slabs as long as 30 ft., combined edge stresses and combined interior stresses of the order of 600 p.s.i. are to be expected under what may be considered average conditions. When the concrete has a higher thermal coefficient and a higher modulus of elasticity than the values used in these computations and when the temperature differential is higher than that assumed, these combined stresses may be greatly increased.

3. In order to effect any significant reduction in combined stresses in the edge and interior of the slab it is necessary to reduce the slab length to about 10 ft. In a slab 10 ft. long and 8 in. thick the combined stresses will be of the order of 400 p.s.i. as compared with 600 p.s.i. in a slab 30 ft. long.

4. The character of the subgrade, as measured by variations in the subgrade modulus between $k = 100$ and $k = 300$, does not have a great effect nor a consistent effect on the magnitude of combined stresses.

SHAPE OF CROSS SECTION OF SLAB

Two types of cross section of the pavement slab are in general use; the cross section of uniform thickness, and the cross section in which the edges of the slab are thicker than the central portion. An appreciable number of State highway departments use slabs of uniform thickness but the majority use the thickened-edge design.

Since the thickened-edge pavement design is used so extensively at the present time, the history of its development is of interest. One of the points of interest in connection with that history, and one which is not generally known, is that, as early as 1912, the California Highway Commission adopted the thickened-edge design as an alternate to a section of uniform thickness for concrete base construction.

In general, two types of thickened-edge cross sections are used. In one, the upper and lower boundaries of the section are parabolic curves so arranged that the thickness gradually increases from a minimum at the center to a maximum at the edge, the edge thickness being from 2 to 3 in. greater than the center thickness. The second type, which is used by a majority of the State highway departments, is the same as that used originally by the California Highway Commission. The central portion of the slab is of uniform thickness and the edge thickness exceeds this by 2 to 3 in. The edge section is a trapezoid, the edge thickening taking place at a uniform rate over the outer 2 to 4 ft. of slab width. In the Arlington tests (15c) it has been found that with this type of cross section the greatest uniformity of load stresses throughout the section may be obtained.

PERMISSIBLE UNIT STRESSES

Before discussing the design of pavement slabs to resist the combined stresses due to load and temperature warping it is desirable to consider the factors that should influence the selection of permissible maximum unit stresses.

As has been stated, consideration of the available data concerning the fatigue limit of concrete has led to the rather general practice of assuming about 50 percent of the ultimate flexural strength as a safe value of the unit stress to be used in designing pavements to resist wheel loads. In general the probable strength of paving concrete at ages greater than 28 days is not definitely known and therefore the design stress has usually been based on the 28-day strength. Since concrete of the character used in pavements may be expected to have a flexural strength at 28 days of from 600 to 700 p.s.i., the customary design stress has been of the order of 300 to 350 p.s.i.

As applied to load stresses this practice is a conservative one and the considerations that lead to this conclusion are:

1. The possibility that the fatigue limit of concrete, for the loading conditions that obtain in pavements, is greater than 50 per cent of the ultimate strength.
2. The possibility that the stresses in pavement slabs caused by impact forces are less than those caused by static loads of the same magnitude.
3. The fact that concrete increases in strength with age and the probability that by the time the pavement has been subjected to enough repetitions of stress due to maximum wheel loads to require consideration of the fatigue limit, the concrete will have attained a strength appreciably in excess of its strength at 28 days.

If the practice of limiting load stresses to about 50 per cent of the 28-day strength of the concrete is a conservative one, then the same practice would certainly be unduly conservative if applied to the design of slabs proportioned to resist the combined stresses due to load and

temperature warping. The additional considerations that lead to this conclusion are:

4. The fact that vehicles having maximum wheel loads constitute a small percentage of the traffic on most roads. The occurrence of maximum stress due to load is therefore relatively infrequent and the occurrence of maximum load stress in combination with maximum warping stress is much less frequent. This is particularly true in those localities where the movement of heavy trucks is principally at night when the warping stresses that are of consequence are generally such that the combined stresses are less than the load stresses.

5. The fact that the unknown stresses due to moisture warping appear to reduce rather than to increase the maximum stresses due to temperature warping.

On the basis of present knowledge the five factors that have been mentioned cannot be definitely evaluated. However, when all of them are considered, it does not appear unreasonable to conclude that, when the design is based on combined stresses due to load and temperature, the safe allowable unit stress is in excess of 400 p.s.i. and may be as high as 500 p.s.i.

DESIGN OF CROSS SECTION FOR COMBINED LOAD AND TEMPERATURE-WARPING STRESSES

A consideration of slab design on the basis of combined load and warping stresses leads to the conclusion that there must be either an increase in permissible unit stresses even beyond the limits that have been suggested or an acknowledgment that current practice with respect to joint spacing in nonreinforced concrete slabs is incorrect.

The curves of Fig. 9 show that, for the 8,000-lb. wheel on dual high-pressure tires, a slab of 9-in. uniform thickness is required if the unit load stress is limited to 350 p.s.i. and that the thickness should be about 8.3 in. if the unit load stress is limited to 400 p.s.i. The combined interior and edge stresses in these same slabs are shown in table 13. It will be observed that the edge stresses are always greater than the interior stresses; that in a 30-ft. slab the edge stresses are equal to or greater than 600 p.s.i.; that in a 15-ft. slab they exceed 500 p.s.i. except when the slab thickness is 9 in. and $k = 100$; and that it is not until the slab length is reduced to 10 ft. that the edge stresses are reduced to values equal to or less than about 400 p.s.i.

Fig. 18 shows the load and warping stresses observed in the Arlington tests in a slab of 6-in. uniform thickness and in a 9-6-9-in. slab. The load in both cases was a static load of 8,000 lb. and the slab length was 20 ft.

The importance of this figure lies in the fact that, from the standpoint both of maximum stress and of uniformity of stress, there is no significant difference between the thickened-edge section and the

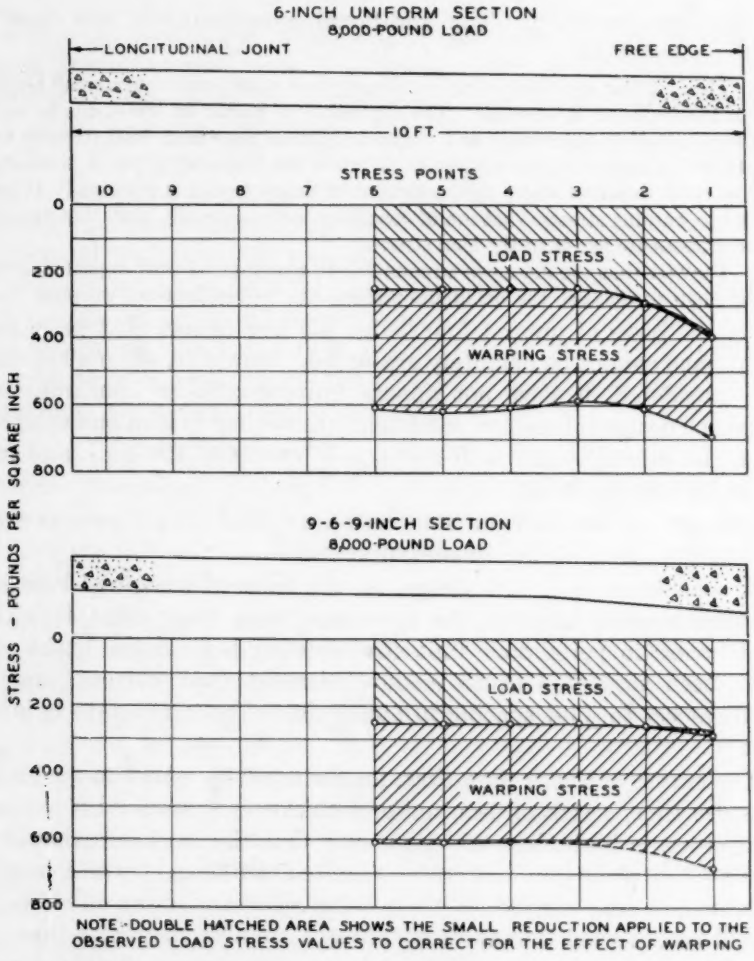

FIG. 18—MAXIMUM STRESS DIAGRAMS FOR COMBINED LOAD AND WARPING STRESSES FOR TWO TYPICAL CROSS SECTIONS. SLAB LENGTH 20-FEET. BASED ON DATA FROM THE ARLINGTON TESTS

section of uniform thickness. The maximum combined stresses are approximately the same for both slabs and the stress diagrams are of approximately the same shape. Therefore, it may be concluded that for long slabs (20 ft. or more) there is no particular advantage, from the standpoint of combined stresses at the edge and interior, of thickening the slab edges. This conclusion does not apply to the slab corners where the load stresses are greatly reduced by edge thickening and where the combined stresses do not exceed the load stresses by any

great amount. With respect to short slabs (length about 10 ft.) a further analysis is necessary before a conclusion can be reached.

One of the basic assumptions of the Westergaard analysis is that the thickness of the slab is uniform. Therefore, neither the Westergaard equations nor the modified equations that have been discussed are strictly applicable to the computation of stresses in thickened-edge slabs. However, by making certain assumptions that appear to be justified, it is possible to compute approximate stresses in a thickened-edge slab that are believed to be reasonably accurate.

Approximate interior and edge stresses, computed on the basis of these assumptions and for the same 8,000-lb. wheel load that has been used in previous examples, are shown in Table 14 for three thickened-edge sections. Also shown in this table are the stresses in slabs of uniform thickness that are approximately comparable, with respect to maximum stress, with the thickened-edge designs. The three pairs of cross sections are designed for maximum combined stresses of approximately 500, 425 and 350 p.s.i.

It will be observed that in all cases, for slabs of this length, the maximum combined stress is less when $k = 100$ than when $k = 300$. The difference is not great in any case and, since the value of the subgrade modulus cannot be predetermined, it is considered reasonable to average the stresses for the two subgrade conditions. On the basis of these average stresses the 9-6-9-in. thickened-edge section is comparable with the section of 7.1-in. uniform thickness; the 10-6.8-10-in. section may be compared with the 8-in. uniform section; and the 11.2-7.8-11.2-in. section may be compared with the 9-in. uniform section.

Since these pairs of slabs are comparable with respect to stress they may also be compared on the basis of probable cost. In making this comparison the depth of the thickened-edge slabs will be assumed to be increased at a uniform rate from the interior thickness to the edge thickness in the outer 2 ft. of slab width. Then in a mile of 20-ft. pavement the amount of concrete required by the slabs of uniform thickness exceeds that required by the comparable thickened-edge slabs by approximately 260, 290 and 280 cubic yards, respectively, for the slabs having uniform thicknesses of 7.1, 8 and 9 in. When consideration is given to the additional expense involved in the construction of thickened-edge slabs, such as shaping the subgrade, shaping joint fillers, the more expensive side forms that are required, and the expense of strengthening the edges of transverse joints, it appears that there is no great difference in cost between the thickened-edge slab and the slab of uniform thickness.

It is concluded that, when pavement slabs are designed for wheel loads such as are commonly permitted by regulatory laws and when the combined stresses due to load and temperature warping are kept within safe limits, the thickened-edge cross section has no marked advantage over the cross section of uniform thickness.

EDGE STRENGTHENING AT FREE TRANSVERSE JOINTS

When a free transverse joint is introduced in a thickened-edge slab, or when a transverse crack develops in a thickened-edge slab that is not reinforced, a condition of relative weakness is created at the edges of the joint or crack. This is because the central portion of the joint or crack has the same thickness as the interior of the slab but is subjected to the higher stresses which are associated with edge loading.

When pavement slabs of uniform thickness are adequately designed to resist edge stresses no edge strengthening at transverse joints or cracks is necessary. When the thickened-edge design is used the edges of joints may be strengthened by methods which are described in the paper. But, when a transverse crack develops in a thickened-edge pavement that is not reinforced there is developed a condition of weakness for which there is no remedy and which may eventually lead to complete failure. This possibility may be avoided by proper design and there are two methods of design available. The first, applicable to nonreinforced pavements, requires the use of a joint spacing of the general order of 10 ft. It is probable that the expense of edge strengthening for so many joints as would be required by this design would lead to the abandonment of the thickened-edge section or the adoption of the second, or alternate, method. The second method is to use properly designed steel reinforcement.

Reinforced slabs can safely be made of any length consistent with the economical use of reinforcement suitably designed to prevent the formation of open cracks. If the design of the reinforcement is such that the stresses to which it is subjected cause either rupture, or excessible elongation, at the cracks which inevitably will develop, then the edge weakness at cracks will not have been remedied. However, if the reinforcement is adequate to hold the edges of the fractured slab in close contact, the crack will tend to act as a hinged joint thereby relieving the warping stresses at the edge and interior, and the interlocking of the irregular surfaces of fracture may be expected to furnish the required edge strengthening along the crack.

The paper then presents a detailed discussion of longitudinal expansion and contraction, the spacing and width of expansion joints, the

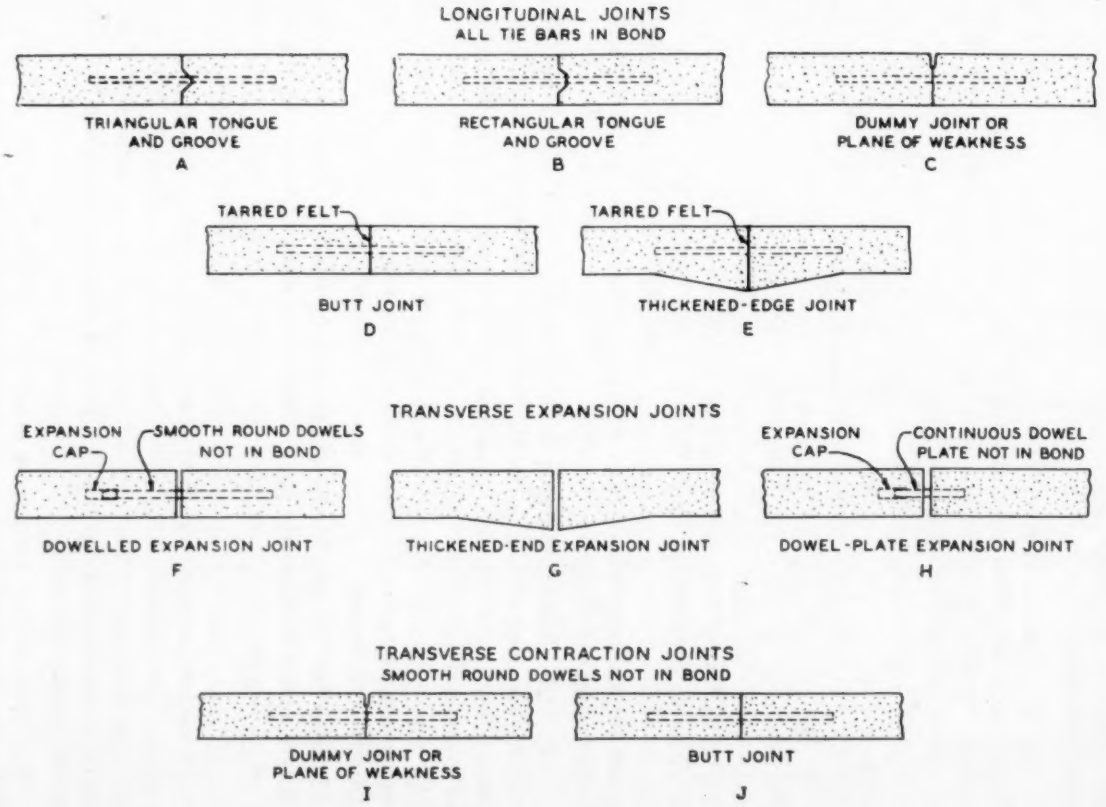

FIG. 22—TYPES OF JOINTS FOR CONCRETE PAVEMENTS

coefficient of subgrade resistance, stresses due to contraction, the design of steel reinforcement, various types of longitudinal and transverse joints, the design of tie bars in longitudinal joints, and the efficiency of joints with respect to transfer of load and reduction of edge stress.

Fig. 22 shows a number of types of longitudinal and transverse joints, most of which were studied in the Arlington tests. The discussion of these joints will be confined here to comment on the thickened-end transverse expansion joint, Fig. 22-G.

In the past the thickened-end type of transverse joint has been criticized on the ground that it offers additional resistance to contraction, with the result that a transverse crack is likely to develop near the junction of the end section with the interior of the slab. No action of this kind has been observed in the Arlington tests. The slabs with thickened ends have expanded and contracted as freely as any of the other slabs tested and no transverse cracks have developed in them in a period of more than 8 years. There is nothing in the results of these tests to indicate that edge thickening cannot be applied to transverse expansion joints with as much success as to the longitudinal edges of the slab.

Very little information of a definite character is available concerning the reported unsatisfactory performance of thickened-end transverse joints. The only reference that has been found is in a 1932 report of a committee of the American Road Builders' Association (43). This report merely states, without giving detailed information, that experience with the thickened-end joint in three States has not been entirely satisfactory.

In contrast to this is the experience of Kent County, Michigan. Otto S. Hess[3] is authority for the following report of that experience.

Since 1926 practically all of the concrete pavements built by the Kent County Road Commission have been constructed with thickened-end transverse expansion joints spaced 50 ft. apart and with no intermediate contraction joints. The 50-ft. slabs are reinforced with wire fabric or bar mats. The expansion joints are ¾ in. wide and a premolded joint filler is used. The ends of adjacent slabs are not connected in any manner.

With this design, transverse cracking has been almost eliminated. Not a single transverse crack has been observed in the vicinity of the joints where the end thickening begins. The contention that contraction in a thickened-end slab will cause the ends to ride up on the

[3]Engineer-Manager, Kent County Road Commission, Grand Rapids, Michigan.

subgrade and create roughness at joints has not been supported since no difficulty has developed due to vertical movement of the slab ends. The experience of Kent County indicates that if the strength required in joint edges is obtained by thickening the slab ends it is not necessary to connect the slabs with dowels or other devices in order to maintain smooth joints.

CONCLUSION

The more important conclusions that are indicated are as follows:

1. The critical load stresses developed in a concrete pavement are primarily dependent on single wheel loads and not on axle loads, axle spacing or the gross weight of vehicle.

2. Impact forces considerably in excess of static wheel loads should be used in the design of pavements. The impact factor (ratio of total impact reaction to static wheel load) is less for balloon tires than for high-pressure tires and decreases as the wheel load increases.

3. The stresses in a concrete pavement are approximately the same for an 8,000-lb. wheel load on dual high-pressure tires and for a 9,000-lb. wheel load on dual balloon tires.

4. The stress analyses of Westergaard, with the modifications suggested by the Arlington tests, are suitable for use in the design of concrete pavement slabs and form the only adequate basis for such design.

5. Since the physical characteristics of the subgrade and of the concrete can never be foretold with certainty it is desirable to be conservative in the selection of values representing these various characteristics for use in design.

6. Warping stresses due to differentials of temperature within the slab may be of the same order of magnitude as the stresses due to heavy wheel loads and therefore require consideration in pavement design.

7. Reasonable assurance of the absence of transverse cracking in concrete pavements can be obtained only by the use of short slabs having lengths not greater than 10 to 15 ft.

8. Transverse cracks in thickened-edge pavements without reinforcement create a weakened condition in the interior of the slab which may be serious. The introduction of properly designed steel reinforcement in long slabs will not completely eliminate transverse cracking but it will reduce or eliminate the detrimental effect of the cracks which may develop.

9. The edges of transverse joints in thickened-edge slabs require strengthening because the central portion of the joint has the same thickness as the interior of the slab but is subjected to the higher stresses that are associated with edge loading.

10. When the pavement is designed for the combined stresses due to load and temperature it is safe practice to use an allowable unit stress in excess of 50 per cent of the 28-day flexural strength of the concrete.

11. When the pavement is designed for maximum legal wheel loads and in such manner that the combined stresses due to load and temperature are limited to safe values and are reasonably well balanced, the thickened-edge section has no great advantage over the section of uniform thickness from the standpoint of over-all cost per mile.

12. Transverse joints are required in concrete pavements to relieve warping stresses due to temperature and also to provide for longitudinal expansion and contraction. Longitudinal joints are required to prevent the longitudinal cracking that usually develops otherwise.

13. If the proper attention is given to the design of both the slab and joint, the required edge strengthening at joints in thickened-edge slabs can be obtained with a number of the types of load-transfer devices in common use.

14. The thickened-end transverse expansion joint is indicated, both by tests and experience, to be a highly effective method of providing the edge strengthening that is required at transverse joints in thickened-edge slabs.

15. Longitudinal joints of the tongue-and-groove type appear to be considerably more effective than other types in common use in providing the strengthening that is required in the edges of the longitudinal joints of thickened-edge slabs.

ACKNOWLEDGMENTS

This paper is essentially a compilation and interpretation of published data and, insofar as practicable, the sources of material are indicated in the bibliography.

The author desires to acknowledge the invaluable advice and assistance given by his associates in the Bureau of Public Roads; L. W. Teller, A. L. Gemeny, J. A. Buchanan, E. C. Sutherland, W. F. Kellermann, R. J. Lancaster and A. L. Catudal.

He also desires to express his appreciation of the generous permission granted by Royall D. Bradbury to appropriate a number of original ideas from his book "Reinforced Concrete Pavements." Special credit is due Mr. Bradbury for originating the simplified methods, used throughout this paper, of computing stresses due to loads and temperature warping. The use of these methods changes a very tedious operation to a very simple one.

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Discussion of a paper by E. F. Kelley:

Application of the Results of Research to the Structural Design of Concrete Pavements*

BY M. G. SPANGLER, A. T. GOLDBECK, AND R. D. BRADBURY

CONVENTION DISCUSSION†

M. G. Spangler‡—This paper by Mr. Kelley marks the beginning of a new epoch in the design of concrete pavements. The first structures of this type were arbitrarily designed, since there were no guiding precedents and no theories available for analysis of the flat slab on a distributed foundation. Along with the accumulation of service experience, research into the structural performance of pavement slabs has been conducted by various workers until a great mass of research data has accumulated. Now, for the first time, as a result of Mr. Kelley's monumental effort toward clarification and correlation of these data, it will be possible actually to design a concrete pavement to meet definitely specified service requirements in much the same manner and with a degree of confidence comparable to that with which engineers habitually design the more orthodox structures, such as bridges, mill buildings, etc.

I desire in this discussion to bring out one or two points in connection with the stress situation in the region of a loaded corner of a concrete pavement slab, both in the light of some research studies which have been made at the Iowa Engineering Experiment Station and the observation of performance of some concrete pavements in central Iowa.

Of the three load situations investigated by Westergaard, Kelley has shown that the corner situation is about as severe as the other

*JOURNAL, Amer. Concrete Inst., June 1939; *Proceedings* Vol. 35, p. 437—presented 35th Annual Convention, New York, March 1-3.

†These discussions, all written, were presented in part at the convention.

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two, even though the effect of warping due to temperature and moisture differentials is less in this region than in the region of an edge or center load. There are many corners built into a concrete paved road; as many as 4,224 in a mile of 20-ft. pavement with a longitudinal joint and transverse joints spaced 10 ft. apart, the number decreasing as the joint spacing increases. In addition, even under the most favorable conditions, transverse cracks may develop at any point sometime during the life of the pavement and each full width crack will add eight more corners to those already built into the pavement. Also, any longitudinal crack which may form will cause additional interior corners wherever it intersects with either a constructed joint or a transverse crack. Thus it is seen that nearly every square yard of a pavement *may* be subjected to the stresses which accompany the application of a load to a corner.

Furthermore, a corner break is one of the most serious types of failure which concrete pavements ordinarily experience. A transverse or longitudinal break does not materially detract from the ability of a pavement to render useful service if it is properly maintained with a bituminous crack filler. On the contrary, a corner break permits the broken off corner to subside due to the reduced bearing area of the subgrade. This greatly increases impact forces on the broken corner, and it may be pounded to pieces within a very short time, thus creating a major maintenance problem and shortening the service life of a pavement. The writer has observed broken interior corners which were completely demolished by traffic within a brief period after the first corner crack appeared. Also, when one of four contiguous interior corners fails, it is only a very short time until the other three corners also fail due to added impact forces and the reduction of support resulting from the failure of the first.

Therefore, speaking broadly, it may be said that because of the large number of potentially vulnerable corners in a pavement and the seriousness of a corner break, the thickness of a pavement should be sufficient at every point to carry a corner load successfully.

Mr. Kelley states that Westergaard's corner load analysis involves the assumption that the load at the corner of a slab is uniformly distributed over a circular area of contact with the circumference of the circle tangent to the edges of the slab, and he bases his criterion for position of the corner load for purposes of design on this interpretation of Westergaard's assumption. I believe that Westergaard's corner load analysis is not limited to loads uniformly distributed over a circular area or that the loaded area must be tangent to the sides of the slab. The analysis is valid for loads distributed in any manner

over an area of contact of any shape, the only restriction being that, since the bending moment is assumed to be symmetrically distributed about the corner bisector, the load must also be symmetrically distributed.

In this respect, the loaded corner of a slab is analogous to a cantilever beam, wherein the maximum moment, which occurs at the beam support, is independent of the distribution of the load and dependent solely upon the magnitude of the total load and the position of the load resultant. The situation is in contrast to that which prevails in the case of edge and center loads, wherein the maximum stress occurs directly under the load and, therefore, the distribution of the load is of vital importance.

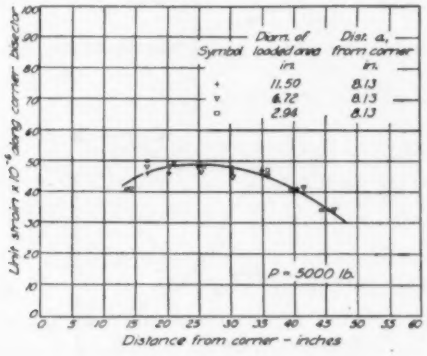
Unit strain x 10⁻⁶ along corner bisector
Symbol
Diam. of loaded area in.
Dist. a_1 from corner in.
+ 11.50 8.13
▽ 6.72 8.13
○ 2.94 8.13
P = 5000 lb.
Distance from corner - inches

FIG. 23

In some tests[1] performed at the Iowa State College by the writer and Prof. F. E. Lightburn, loads were applied to a slab corner over three circular areas, having diameters of 2.94 in., 6.72 in., and 11.50 in., respectively. In each case the center of pressure was applied on the bisector at a distance $a_1 = 8.13$ in. from the corner. The measured strains in each of these loadings was the same, as shown in Fig. 23. From these facts it is apparent that the *position* of the resultant of the load at a corner is the important element to be determined.

In my opinion it would be more logical to determine the distance a_1 for purposes of stress calculation from a study of the path which loaded wheels travel in the region of a corner and locate the point of intersection of this path with the corner bisector, rather than determine the distance a_1 from the area of contact of the wheel load.

[1]Spangler, M. G. and F. E. Lightburn. Stresses in Concrete Pavement Slabs. Proc. Highway Research Board, Vol. 17, p. 215-230. 1937.

Prior to 1935, the writer observed and reported[2] that corner breaks in concrete pavements seemed to vary in shape between a straight line normal to the corner bisector and an arc of a circle having the slab corner as a center. The shape of some typical corner breaks observed at Ames are shown in Fig. 24. Also, some laboratory stress measurements indicated that the locus of maximum stress across a corner region was a curved line concave toward the corner, and that the stress was not uniformly distributed over this locus, but was greater in the

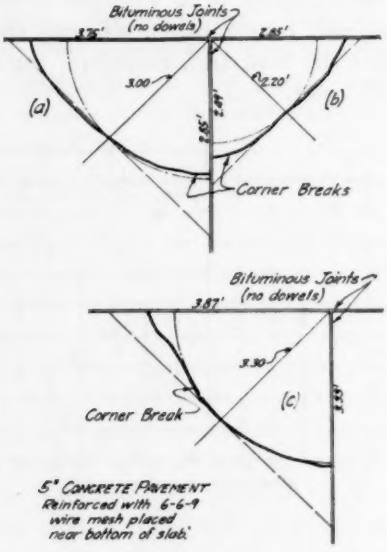

FIG. 24

vicinity of the corner bisector than at the edges of the slab. On the basis of these observations the writer proposed two stress formulas, one for the minimum probable stress, assuming the moment to be uniformly distributed over a line normal to the bisector, which is the assumption Westergaard made, and one for a maximum probable stress, assuming the moment to be non-uniformly distributed over the arc of a circle. The minimum probable stress formula was similar to Westergaard's corner formula, except that it was stated in a general form for any point on the corner bisector, whereas Westergaard had differentiated the expression to obtain a formula for stress at one point only. The maximum probable stress formula gave stresses 53 per cent greater than the minimum, and the hypothesis was stated that

[2]Spangler, M. G. Stresses in Concrete Pavement Slabs. Proc. Highway Research Board, Vol. 15, p. 122-146. 1935.

the actual stress in a slab probably would lie somewhere between these extremes. Laboratory measurements of stresses in one slab showed values nearer to the maximum stress curve, whereas in another experimental slab the measured stresses were nearer the minimum.

The formula 7a announced in Mr. Kelley's paper gives stresses between these minimum and maximum values, being about 30 per cent greater than the minimum when $a_1 = 5.65$ in. and about 46 per cent greater than the minimum when $a_1 = 11.31$ in. within the range of slab thickness from 7 in. to 11 in. Formula 7a would appear to be satisfactory as a design formula, therefore, in the light of the above mentioned hypothesis. However, since formula 7a is purely empirical and simplification is both desirable and permissible, the fractional exponent of the quantity $\left(\frac{a_1}{l}\right)$ may be eliminated to advantage and the numerical coefficient of the expression raised to 3.2, which gives $\sigma_c = \frac{3.2\ P}{h^2}\left(1 - \frac{a_1}{l}\right)$, as shown in Fig. 25 and 26. This simplified expression gives stresses very close to those of formula 7a, and its use in place of the B.P.R. formula is recommended solely because of its simplicity.

In regard to the safe allowable working stress, Mr. Kelley concludes on page 73c, that "..... when the design is based on combined stresses due to load and temperature, the safe allowable unit stress is in excess of 400 p.s.i. and may be as high as 500 p.s.i." I would not disagree with this conclusion when applied to center and edge loading conditions, where the temperature and warping stresses are a relatively large part of the combined stresses, but for the corner load condition, the safe allowable combined stress probably should not exceed 400 p.s.i., and the safe allowable stress due to static and impact wheel loads alone should not exceed 350 p.s.i. This conclusion arises from a consideration of the following factors:

1. In the corner load situation, nearly all the stress induced in the slab is attributable to wheel loads plus impact. Maximum corner stresses are, therefore, more positive and frequent than edge and interior stresses.

2. A corner break is the most damaging type of failure to which pavements are liable.

3. Although concrete of the character used in pavements may be expected to have a flexural strength of from 700 to 900 p.s.i. at one year, these are average strengths, and many small volumes such as

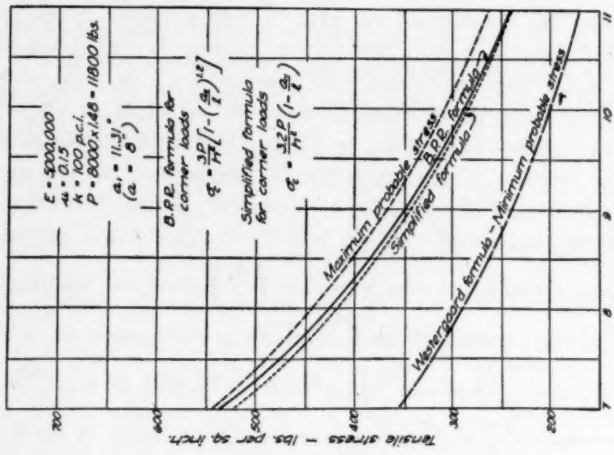
E = 3000,000
μ = 0.15
k = 100 pci.
P = 8000 x 1.48 = 11800 lbs.
a = 11.3"
(a = 8")
B.P.R. formula for corner loads
q = 3P/πh² [1-(√4a/l)^1.2]
Simplified formula for corner loads
q = 3.2P/h² (1-√4a/l)
Maximum probable stress
B.P.R. formula
Simplified formula
Westergaard formula = Minimum probable stress
Tensile stress - lbs. per sq. inch.
Thickness of slab - inches.

FIG. 26

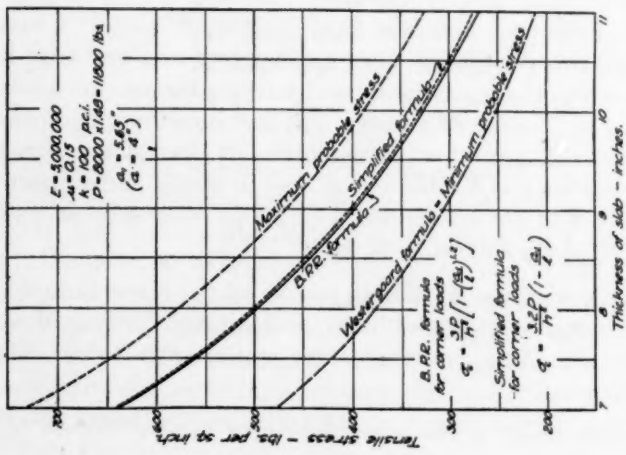
E = 3000,000
μ = 0.15
k = 100 pci.
P = 8000 x 1.48 = 11800 lbs.
a = 5.65"
(a = 4")
Maximum probable stress
Simplified formula
B.P.R. formula
Westergaard formula = Minimum probable stress
B.P.R. formula for corner loads
q = 3P/πh² [1-(√4a/l)^1.2]
Simplified formula for corner loads
q = 3.2P/h² (1-√4a/l)
Tensile stress - lbs. per sq. inch.
Thickness of slab - inches.

FIG. 25

are involved in corner breaks, may be expected to have less than the average strength. This probable deviation from the average may easily amount to as much as 25 per cent.

4. There always has been and probably always will be a certain amount of overloading of pavements beyond the legal weight restrictions. The amount and frequency of such overloading cannot be estimated, but its inevitableness suggests that a larger factor of safety is desirable in the corner region, where wheel loads are the principal stress producing agency.

Mr. Kelley states the conclusion that the thickened-edge cross section has no marked advantage over the cross section of uniform thickness. I would go further than this and say that, in my opinion, a uniform cross section is superior to a thickened-edge cross section, largely because of the greater and more positive strengthening of the interior corners and edges at transverse and longitudinal joints which the uniform cross section affords.

The earliest pavements were constructed with a relatively thin uniform cross section, or in a few cases, with the edges thinner than the center. Since these early pavements had no center longitudinal joints, there were no interior corners in the pavement as constructed. Also, the exterior corners were only feebly supported by the section on the other side of a construction joint, and many corners broke off under traffic. This experience led to the adoption of the thickened-edge cross section, primarily to strengthen the exterior corners.

Later, the adoption of the longitudinal center joint introduced many interior corners into the structure, and, while it was believed that dowel bars and other load transferring devices, and corner reinforcement bars would support these interior corners and protect them from damage, service experience has shown that this protection has not been adequately realized. There are many examples of failures of one or more interior corners formed where the longitudinal and transverse joints intersect, even in thickened-edge pavements with interior thickness greater than $\frac{2}{3}$ the edge thickness, a ratio not infrequently recommended.

In Iowa, several thousand miles of pavement having a 10-7-10 cross section were constructed between about 1928 and 1933. It was found that after 7 or 8 years of service, many interior corners began to fail on the heaviest traveled routes. Apparently these interior corners were strong enough for a limited number of applications of load, but as the number of stress repetitions increased, the corners began to fail. A typical example of this type of failure is shown in Fig. 27 before maintenance replacement of the broken corners. In 1936,

FIG. 27—TYPICAL INTERIOR CORNER BREAK ON U. S. HIGHWAY 6, EAST OF NEWTON, IA.

as a result of this experience, the thickness of the central portion of the standard cross section was increased to 7½ in. While it is too early in the life of these thicker pavements to observe the effect of this increased thickness, we may analyze the probable effect in the light of the latest knowledge of corner load stresses.

According to the simplified corner formula, and neglecting both warping stresses and whatever support was available from dowel bars and corner reinforcement bars, the stress in the 7-in. slabs near an interior corner must have been about 525 p.s.i. under a legal wheel load concentration of 8,000 lb. plus 3,800 lb. impact increment, applied at a distance 11.3 in. from the corner. Increasing the thickness to 7½ in. would tend to decrease the stress to about 465 p.s.i., which is still too high, and it seems reasonable to expect that these thicker pavements will begin to develop interior corner breaks after being in service about 10 to 12 years on the heavier traveled routes. To decrease the interior corner stress, due to wheel loads only, to a reasonably safe working value of about 350 p.s.i. it would be necessary to increase the interior thickness to about 9 in.

The exterior corners of these pavements have apparently been strong enough. The writer has never seen an exterior corner failure on any of the pavements in central Iowa where numerous interior corner breaks have been observed. While there is no method whereby we may calculate the strength of exterior corners of a slab having a thickened-edge cross section, it seems entirely probable that the exterior corners of a 9-in. uniform slab would be at least as strong as those of the slab having a 10-7-10 cross section. Therefore, from these observations the conclusion is reached that a uniform cross section

about 9 in. thick is both necessary and adequate to carry traffic having a legal 8,000 lb. wheel load limit, for a long period of service.

Such a design would probably have a factor of safety for an unlimited number of stress repetitions which would average slightly greater than one, although due to the normal variation in concrete strength, there would probably be some individual corners with a factor of safety of one or less. The strength of the slab at interior corners and along transverse and longitudinal joints would be independent of any type of load transfer devices, tongue and groove joints, corner reinforcement bars, etc. The function of dowel bars would be limited to the maintenance of slab alignment and the prevention of faulting at the joints.

*A. T. Goldbeck**—In the few days during which Mr. Kelley's paper has been at my disposal I have been unable to read it thoroughly, much less study its one hundred and thirty-five pages, seventeen tables and twenty illustrations. Consequently, the present discussion has many shortcomings.

It is my opinion that Mr. Kelley has done an admirable job of gathering up the threads of information which diligent research and analysis have produced during the past twenty or more years. The collection of this information under a single cover is an extremely valuable contribution to the theory of concrete highway design.

The design of a concrete road from a theoretical standpoint is an extremely difficult and complicated problem. Especially is it difficult to decide what combination of circumstances may occur simultaneously to produce maximum stresses. Mr. Kelley has attempted to consider the various stress-producing influences, not only separately, but in their relationship one to the other, to the end that critical stress may be determined.

He has brought out the fact that impact is a factor to be reckoned with but I wonder if this same conclusion would be reached from actual stress measurements on concrete roads under moving loads. As I understand it, thus far impact pressures have been measured and deformation in beams under impact have been determined. I have in mind that when impacts are delivered to a highway, the motor truck is generally moving at high speed. If the truck is moving at 40 miles per hour, or 60 feet per second, in 1/100 of a second the central point of contact between the tire and the road has moved 0.6 feet. Does it not seem likely that impact delivered under these circumstances

*Engineering Director, National Crushed Stone Assn., Washington, D. C.

may not be as severe in stress-producing effects as impact delivered at a given point? It would be my suggestion that this possibility be investigated by taking stress measurements on actual highways subjected to moving vehicles.

In connection with the fatigue of concrete and the application of the value for modulus of rupture in determining the safe unit stress for use in design, it will be recalled that the fatigue tests of the Illinois Division of Highways were made on cantilever beams. The present standard method for making tests for modulus of rupture involves the use of a simple beam with third point loading and this method gives results which are about 0.8 to 0.85 in value of the results given by the cantilever beam method of loading. Consequently, if the modulus of rupture as determined by the third point loading method is used in determining the safe unit stress, a more conservative design will result than is the case when the cantilever beam is used. Furthermore, what is the relation between modulus of rupture as we now determine it and the modulus of rupture value which applies under the loading conditions in a concrete road slab. If two different kinds of loading, cantilever and simple beam with third point loading. give different results, the foregoing question is a very natural one to ask. It, of course, is something we know nothing about. It introduces an uncertainty.

I am quite in agreement with Mr. Kelley's conclusion that "While there is justification for the present practice of assuming the design stress to be approximately 50 per cent of the ultimate strength, the many uncertainties regarding fatigue characteristics make any greater refinement in the fatigue data of doubtful value."*

With regard to formulas for stress produced by wheel loads, I think those of us who originally, some twenty years ago, were guilty of trying to make use of the principles of mechanics to learn something of the stress conditions, did so with no thought in mind of any great exactness. Certainly, the design of a concrete road slab does not lend itself to exactness, because of the many variable conditions and combinations which are bound to exist. The corner formula, involving a single point loading applied at the very tip of the corner, was considered approximate as a matter of course. None the less, it was useful and still is useful for rough investigation and I still like to use it in its very simplist form.

I must confess that one of the startling phases of Mr. Kelley's paper is the somewhat high stress produced in the interior of the slab under

*This quotation is not included in the digest of Mr. Kelley's paper as published in this JOURNAL among "the more important conclusions."—EDITOR

loading, the original Westergaard analysis, especially, giving high stress results. I do not know that I can agree with Mr. Kelley's conclusion that in spite of the apparently high results obtained by equation 8 in the original Westergaard analysis, it will be conservative and not uneconomical to continue to use the results given by this formula. I do not see how the stress in the interior of the slab is highly important in any event, since at almost any time, the point under consideration may become an edge due to the formation of a transverse crack.

I think that perhaps one of the phases of Mr. Kelley's paper which will be surprising to many, deals with the question of warping stresses. Warping stresses are really bending stresses and are shown to be very high, particularly when the slabs have lengths to which we are accustomed, for instance, 30 feet. As the value for K, the modulus of subgrade reaction which measures the stiffness of the subgrade, increases, apparently the warping stresses increase and the indication seems to be that, for short slabs at least, subgrade treatment may be harmful under concrete roads because of the hardness of the subgrade increasing the warping stress.

It is my opinion that we should not jump too readily at this conclusion, without at the same time considering the possible beneficial effect of subgrade treatment. Unquestionably much bending stress is produced in concrete slabs due to the continuously changing support offered by the subgrade. It is a well-known fact that the tendency is for concrete roads to become rougher with age and this can be due to nothing but the changing support offered by the subgrade. Certainly, with some types of subgrades, this may be a far more important factor in the production of cracking than warping stresses, or even of traffic loads. I am not convinced that the results obtained from the formulas used for calculating warping stresses are entirely correct. In this connection, let me refer to Table 9* in which are shown transverse temperature warping stresses. The effect of transverse warping is to produce bending of the slab transversely. It would seem that the greatest transverse stress which could possibly exist would be that in which the slab is so warped that it rests upon a line of support at the center of the slab. The dead load stress for a ten-foot width of a 6 in. slab thus supported is 156 lb. p.s.i., whereas the warping stress for a modulus of subgrade reaction of 300, is given at 210 lb. p.s.i. For an 8-inch slab the maximum possible dead load stress at the center, assuming the slab supported at the center and overhanging at the end,

*Table 9 does not appear in the brief version of Mr. Kelley's paper published in this JOURNAL.—EDITOR

is only 117 lb. p.s.i. as against a calculated warping stress of 190 lb. p.s.i. This does not seem logical to me and I am wondering if the warping stresses in a longitudinal direction are really as high as the calculations show. When warping is produced, there must have been a recent change in temperature, for warping can only result when there is a difference in temperature between the top and the bottom of the slab. If the top of the slab is cooler than the bottom, there probably has been a fall in temperature which has resulted in dragging the free ends of the slab over the subgrade. Under these conditions the tension in the top of the slab produced by warping would probably be increased to some extent by the frictional effect, but, on the other hand, under the action of loads this tension would be reduced along the edge and in the interior of the slab. I wonder if these warping stresses should not be corrected to take account of the fact that a horizontal force exists at the bottom of the slab because of subgrade friction.

If the top of the slab is hotter than the bottom, there probably has been a recent rise in temperature of considerable amount which has resulted not only in warping, but also in a general expansion of the slab longitudinally. Would this not tend to produce high direct stress in compression which would relieve some of the tension in the bottom due to warping? I note that warping stresses were observed on a test slab at Arlington. It would be very interesting to me to know how such stresses were observed. It is obvious that if deformations are taken with a strain gage, such deformations might give misleading results. It is perfectly possible, for illustration, to have high stress where no deformation is observable whatever; and, correspondingly, have zero stress in spite of what seems to be a deformation. I realize, of course, that these points are entirely clear to the investigators of the Bureau of Public Roads, but they are well worth emphasizing, none the less.

It, no doubt, will be interesting to many highway engineers to learn of Mr. Kelley's conclusion regarding the thickened edge. The thickened edge resulted primarily from an analysis of pavements from the standpoint of their ability to carry concentrated loads. The extremely high longitudinal warping stresses, which Mr. Kelley's analysis emphasizes, are no doubt primarily responsible for his conclusion regarding the thickened edge. In his discussion of non-reinforced pavements, he points to the eventuality of such pavements cracking so frequently that joints not more than ten feet apart would be required to avoid this difficulty. Here, again, warping stresses come into the picture. It is hard, however, to reconcile such short

spacing of joints with the fact that there are many miles of concrete roads having crack spacing on an average of 90 feet by actual survey and these roads extend in age up to nine years.

In connection with the matter of subgrade friction which was recognized as an important stress-producing force many years ago, I would call attention to the fact that during the first day or two of hardening of the concrete there frequently are large changes in temperature, bringing about contraction of the concrete while it is still in a very green condition and having exceedingly low resistance to direct tension. We have all seen transverse cracks in concrete pavements the day after they have been laid. This force of friction cannot be overlooked at early periods, nor can it be overlooked in its combination with warping effects at later periods.

I shall leave the discussion of reinforcement to others.

Again, let me express my very sincere appreciation of the splendid paper which Mr. Kelley has written. I do not think the subject of concrete road design is by any means exhausted. Additional researches should be made on actual impact stress under rolling loads; on the effect of variable subgrade support; and the effect of subgrade characteristics on stress. Plastic flow of concrete in tension certainly needs study, as does also the question of the real meaning of modulus of rupture in terms of the kind of stress actually produced in highways. This stress may be partly bending and partly direct tension. We should learn more about the relationships of modulus of elasticity, coefficient of expansion and conductivity because of their effects on stress. Finally, more intensive study should be given to the determination of those combinations of phenomena which produce additive stresses and those which have a stress neutralizing effect. This latter study is one of the very highest significance, for upon it rests the rational design of the concrete road.

I present the foregoing remarks in a spirit of admiration and in an attempt at friendly helpfulness.

*R. D. Bradbury**—Mr. Kelley has made a valuable and timely contribution to the subject of concrete pavement design. In reading Mr. Kelley's paper, one is impressed with the high stress-producing possibilities of the external conditions to which all concrete pavements are exposed. In analyzing these maximum stress possibilities, the author has confined his attention to stresses induced only by load and by temperature. This has been necessary, since these are about the only important external conditions that can be translated into

*Engineer-Director, Wire Reinforcement Institute, Washington, D. C.

stress effect, either by computation or by field measurement, with any reasonable degree of confidence. Nevertheless, it must be realized that, in addition to traffic loads and variations in temperature, concrete pavements are also subjected to other stress-producing conditions such as variations in moisture content, non-uniform settlement of the subgrade, and even volumetric changes in the subgrade itself whereby the subgrade may cease to act merely in a passive manner to the extent of the reactions induced in it and may itself become an agency of active force application.

When one realizes, as the author has so clearly pointed out, that certain combinations of load and temperature alone may induce stress intensities of the order of 500 to 600 p.s.i., and possibly more, it is apparent that, if attempt be made to design concrete pavements so conservatively as to hold maximum stress possibilities within the limits of a safe modulus of rupture strength of the concrete as predicated upon unlimited stress repetition, resulting designs will be so thick or will require so many joints as to render the concrete pavement prohibitive in cost.

As the author has shown, thickness has its distinct limitation in preventing the development of high combined stresses. All that can be done to prevent these high combined stresses is to reduce that part of the stress which is contributed by differential temperature by reducing the length and width of the slab. But in order to accomplish any substantial reduction, even in this part of the combined stress, it is necessary to resort to slab units whose horizontal dimensions must not exceed 10 or 12 feet. And that means the use of some 400 to 500 transverse joints per mile which would be highly undesirable for purely utilitarian reasons to say nothing of cost and upkeep.

Because of these restrictions upon the dimensions of slab units—as are necessarily imposed by considerations of desirability and economy —and in view of the dubious ability of such units to resist safely at all times the uncertain, but undoubtedly high, stress-producing possibilities of their exposure, it is quite apparent that the structural design of concrete pavements must, in the light of present knowledge and experience, be predicated upon the probability of erratic cracking, regardless of any dimensional proportions that may be provided within limits of reasonable economy. From the viewpoint of practical design, it, therefore, becomes necessary to render expectant cracks structurally harmless by properly controlling the character of their subsequent behavior, rather than to attempt to preclude all possibility of their occurrence. Thus, it is generally recognized that the basic objective

in the structural design of concrete pavements is to attain satisfactory crack control rather than to attempt actual crack prevention.

This, then, permits the use of a liberal working stress for the concrete in flexure in order to obtain economical thickness and a reasonable joint spacing. And, even though such sections may be susceptible to erratic cracking at a relatively early age, still they can be economically protected against loss in load-carrying capacity—when, as and if cracks do occur—by providing a moderate amount of well-distributed reinforcement to hold crack faces in tight interlock when adjoining slab sections contract, thereby rendering such cracks capable of transferring load and giving them at least as much protection as can be given to the doweled edges at transverse joints.

That only a moderate amount of steel will serve this purpose is due to the fact that it is used not to increase beam-strength of the slab but merely to hold crack faces in tight interlock so that the two crack edges will deflect simultaneously rather than independently as a load passes over the crack. All that is needed, then, is sufficient steel to resist adequately the force developed by subgrade friction throughout half the free length or width of the slab unit when contraction occurs. The rational design of reinforcement for concrete pavements thus involves determination of the external forces that are created by virtue of the frictional contact that exists between the slab and its supporting subgrade. In practice, the intensity of this frictional force is commonly expressed as a coefficient times the weight of the slab per unit area of contact.

Selection of an appropriate value for the frictional coefficient must necessarily be based upon the results of tests of the so-called force-displacement type, whereby a slab unit of known weight resting upon a given subgrade is moved or displaced laterally with reference to the subgrade, thereby revealing the magnitude of thrusting force required to produce successive increments of slab displacement. The results obtained from experimental investigations of this character have been reported in detail by the United States Bureau of Public Roads.(1) All such tests have consistently shown that the magnitude of thrusting force required to produce displacement of the slab increases as the amount of slab movement is increased up to some maximum value, beyond which sliding of the slab occurs without developing any further increase in frictional resistance. Maximum values of the frictional coefficient, as revealed by test, are approximately as shown in Table 15.

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[1] *Public Roads*, Nov., 1935.

TABLE 15—MAXIMUM VALUES OF THE COEFFICIENT OF SUBGRADE FRICTION, C_m, AS INDICATED BY FORCE-DISPLACEMENT TESTS

4-inch slab, maximum C_m = 3.1
6-inch slab, maximum C_m = 2.5
8-inch slab, maximum C_m = 2.2

In order to apply these experimental data appropriately in practice, it is necessary to take into account the fact that the lateral movements that actually occur in pavement slabs are not produced by the application of external thrusting forces. They are merely the changes in slab length and width resulting from lateral expansion or contraction of the slab, such as may be caused, for example, by a change in temperature. Obviously, any movement developed in this manner is not a bodily movement of the slab as a whole, but is merely an accumulated deformation with respect to the slab center which necessarily remains stationary. Hence, the actual displacement of the slab relative to the subgrade can not be the same at all points throughout its entire length but must vary from zero at the center to a maximum at each free end, the amount of displacement at any point being directly proportional to the distance of that point from the center of the slab. The value of the frictional coefficient appropriate for use in practice, therefore, is not the maximum possible value, as revealed by the force-displacement test, but an effective value averaged over half the slab length and consistent with anticipated slab movement.

For any given set of conditions, the average value of the frictional coefficient is governed, of course, by the characteristics of the force-displacement curve as determined by test. But, in view of the consistent shape of such curves, it is believed that a certain regularity of variation may be assumed without appreciable error in order to render the basic properties of force-displacement relationship analytically adaptable to the varying design requirements that occur in practice. The force-displacement tests previously referred to reveal the fact that approximately 90 per cent of the absolute maximum value of the frictional coefficient is developed at a movement of about .06 inches. This fact, considered in conjunction with the observed character of variation at smaller movements, suggests the permissibility of assuming, for practical purposes of design, that the frictional coefficient develops its maximum value with any movement in excess of .06 inches and increases from zero to the maximum as the ordinates to an inverted second-degree parabola having its vertex at the center of the slab. As illustrated by the superimposed comparisons shown in Fig. 28 for both a 6-in. and 8-in. slab, it is seen that a force-displacement curve assumed to vary in this manner is not only conservative in each case but conforms very closely to the actual curve as determined by test.

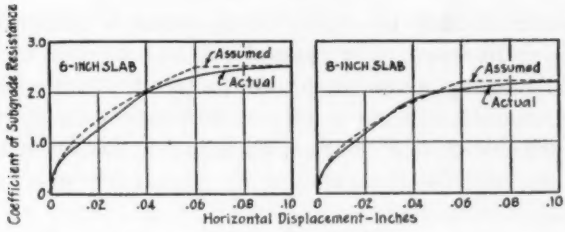

FIG. 28—COMPARISON OF ASSUMED AND ACTUAL FORCE-DISPLACEMENT CURVES

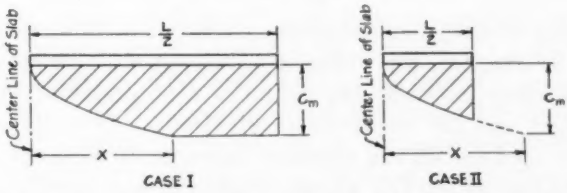

FIG. 29—VARIATION IN VALUE OF THE COEFFICIENT OF SUBGRADE FRICTION FROM THE CENTER TO THE END OF A PAVEMENT SLAB

Based on this assumed variation for the force-displacement curve, an average coefficient C may be expressed in terms of the maximum coefficient C_m, the slab length L, and a quantity X which is the distance from the center of the slab to the point where the change from parabolic variation to a constant value occurs. As indicated in Fig. 29, two cases arise, depending upon whether half the slab length is greater or less than X. For each case, the average coefficient is expressed by one of the following formulas:

$$\text{Case I, } X < \frac{L}{2} \qquad C = C_m \left(1 - \frac{2X}{3L}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

$$\text{Case II, } X > \frac{L}{2} \qquad C = \frac{2C_m}{3}\sqrt{\frac{L}{2X}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

in which, C — average value of the coefficient of friction
C_m — maximum value of the coefficient of friction
T — temperature change in degrees Fahrenheit
L — free length of slab when computing forces in a longitudinal direction; or free width when computing forces in a transverse direction.
X — distance from center of slab to the point where the intensity of frictional force changes from parabolic variation to a constant value.

The distance X may be evaluated in terms of the condition that causes the slab to move. For example, if, as indicated in Fig. 28, the transition is assumed as occurring at the point where a displacement of .06 in. is attained and the condition that causes the slab to move is a temperature change of T degrees, then, taking the thermal coefficient of concrete as .000005, the value of X, when expressed in feet, must satisfy the equation,

$$.000005 \times T \times 12X = .06$$

from which,

$$X = \frac{1000}{T} \text{(ft.)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

In evaluating the average subgrade coefficient in this manner for any given case in practice, the width of pavement and the chosen spacing of free transverse joints fix the values of L to be used respectively in the transverse and the longitudinal directions. As to the appropriate value to be used for T, the question arises as to whether or not this should be taken as the full annual change in temperature. It would seem that, to do so, is unnecessarily conservative.

Suppose for example that, after being deformed laterally, the subgrade material were devoid of any appreciable recovery or "spring-back" properties. In that case, no force of subgrade friction could exist except by virtue of a sustained movement of the slab. The instant that slab movement ceased, the stresses induced in the slab would cease to exist, elastic recovery of the steel (or the concrete if unbroken) would draw the slab back, and a fresh cycle of stress development could not begin until a condition of changing temperature is again resumed. In other words, an appropriate value for T would be the maximum change in temperature that may reasonably be expected to occur in the nature of an uninterrupted periodic change rather than the total change that might accumulate during the interval from the hottest period of summer to the coldest period of winter.

Although the force-displacement tests previously referred to indicate that the non-granular types of subgrades may possess the ability to recover as much as 30 per cent of their maximum lateral deformation upon release of the applied thrusting force, still it is inconceivable that, even in such cases, any appreciable retained reaction against the slab could be continuously conserved throughout the interval of time required to develop the maximum yearly change in temperature. The maximum yearly change is not developed in the nature of a continuous drop from the yearly high to the yearly low. It represents the net accumulated change resulting from the plus and minus increments that occur almost daily. In other words, nearly every day

certainly every few days—there is always some rise from a previous low, and such a rise, even in those cases where a resilient type of subgrade is involved, would only have to be about one-third as much as the previous drop in order to largely, if not entirely, counteract the recoverable portion of the subgrade deformation previously developed. It is, therefore, reasonable to conclude that a temperature increment approximately equal to a liberally-estimated maximum daily change, or say about $T = 40°$, is ample as an average value for general use in applying the above formulas.

Based upon maximum values for C_m as given in Table 15 and using a temperature change of $T = 40°$, values for the average coefficient C obtained by application of Formulas (1), (2) and (3) are as shown in Table 16. Thus, it is observed that, in applying the so-called "subgrade-drag" theory to the design of reinforcement for concrete pavement slabs of thicknesses and lengths as commonly built, it is seldom necessary to use an average value for the frictional coefficient greater than about 1.6 in designing the longitudinal steel; whereas a value of about 1.0 to 1.2 is usually adequate in designing the transverse steel.

TABLE 16—VALUES OF THE AVERAGE COEFFICIENT OF SUBGRADE FRICTION, C, SUITABLE FOR GENERAL USE IN DESIGNING REINFORCEMENT

Thickness of Slab (Inches)	Free Length of Slab				
	20 ft.	30 ft.	40 ft.	50 ft.	60 ft.
4	1.31	1.60	1.85	2.07	2.27
5	1.14	1.40	1.62	1.80	1.98
6	1.06	1.30	1.50	1.67	1.83
7	.97	1.19	1.38	1.53	1.68
8	.93	1.14	1.32	1.47	1.61
9	.89	1.09	1.26	1.40	1.54

Based on a temperature change of 40°F.

Design of Reinforcement—Illustrative Example

Design the reinforcement required for a concrete pavement, 8 inches thick (weighing 100 lb. per sq. ft.), with free transverse joints spaced 40 feet apart. Allowable stress in wire fabric 28,000 lb. p.s.i.

Longitudinal Steel—From Table 16, for an 8-in. slab, the value of the average frictional coefficient in the longitudinal direction ($L = 40$ ft.) is $C = 1.32$

On a strip one foot wide, the total longitudinal force developed by subgrade friction in half the slab length is,

$$100 \times 1.32 \times \frac{40}{2} = 2640 \text{ lb. per ft. of width}$$

and the required sectionl area of steel, at 28,000 lb. per sq. in., is,

$$A_s = \frac{2640}{28000} = .094 \text{ sq. in. per ft. width of pavement}$$

which is provided by,

No. 3 gauge wires, spaced 6 in. c/c (giving $A_s = .093$)

Transverse Steel—From Table 16, for an 8-in. slab, the value of the average frictional coefficient in the transverse direction ($L = 20$ ft.) is $C = 0.93$

Hence, the force developed by subgrade friction on a unit strip in the transverse direction is,

$$100 \times 0.93 \times \frac{20}{2} = 930 \text{ lb. per ft. of length}$$

and, transversely,

$$A_s = \frac{930}{28000} = .033 \text{ lb. per ft. length of pavement}$$

which is provided by,

No. 5 gauge wires, spaced 12 in. c/c (giving $A_s = .034$)

Resulting Design—Wire fabric having No. 3 gauge longitudinal wires spaced 6 inches c/c, and No. 5 gauge transverse wires spaced 12 inches c/c; or, as commonly written,

Welded wire fabric 6″ × 12″ mesh, No. 3 and No. 5 wires

which, based upon sheets 9′-6″ wide, weighs approximately 45 lb. per 100 sq. ft.

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The Bleeding of Portland Cement Paste, Mortar, and Concrete*

Treated as a Special Case of Sedimentation

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MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

The bleeding of paste, mortar, and concrete is identified as a subsidence or sedimentation of the temporarily suspended solids. A method of measuring such subsidence is given. The bleeding of a given sample occurs in two stages; a period of constant rate of subsidence, followed by a period of diminishing rate. The rate of subsidence during the constant period was shown to follow a general law derived from Poiseuille's law of capillary flow; hence its magnitude depends upon the hydraulic gradient and the size of the capillary channels between the suspended particles. The size of channels depends on the specific surface of the solids, on the water content of the mix, and on a constant which is characteristic of the solids in suspension, and which accounts for the quantity of water which remains immobile with respect to the solids. Examples are given showing the application of the law to the control of bleeding rate.

Bleeding capacity is defined as the amount that a given mix can subside per unit of its original height when the effect of setting does not interfere. It was found to obey a law derived on the assumption that all particles move vertically downward with no lateral motion whatever. An ideal mix (from the standpoint of bleeding effects) is one in which its bleeding capacity is determined by the bleeding capacity of the paste, together with the paste content. A faulty mix exhibits premature bridging of the coarser particles and thus a lack of cooperation between the paste and the aggregate. The influence of premature bridging on quality is discussed.

*Presented by the author at the 35th Annual Convention of the A. C. I. in New York, March 1-3. This is a brief version only of an extensive paper which will shortly be available as a publication of the Portland Cement Association.

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This is a brief summary of a part of a study of bleeding which has been under way in the Research Laboratory of the Portland Cement Association, Chicago. The study is fully reported in an extensive paper which is the basis of this summary. Of the many phases of the problem that are treated in the complete paper, only a few can be presented here; it is therefore impossible to convey a complete picture of the subject in the space available.

Like other investigations of this subject, this one was inspired by observations of the relationship of bleeding to "water gain," interference with satisfactory finishing of slabs, low degree of plasticity, and paste stratification, that is, the internal changes which result in non-uniform bond between the paste and aggregate. Besides these commonly observed detrimental effects, the study was made with certain beneficial effects in mind also, for it was believed that a knowledge of the fundamentals of the process of bleeding was essential to the proper utilization of "forced bleeding;" that is, the consolidation of concrete by absorbents or by the newly developed vacuum method.

The discussion which follows is restricted to a consideration of the phenomenon which results in an accumulation of clear water which takes place slowly by uniform seepage over the entire surface of the freshly placed concrete after all placing operations have ceased. The effects of puddling, vibration, or other intermittent irregular procedures are not considered; their qualitative influence can be inferred after the principles governing the basic phenomenon in its simplest aspects are understood.

TEST METHOD

The method of obtaining the data is illustrated in Fig. 1 and 2. In this particular case the vessel contains a sample of cement paste which is covered with a layer of water added immediately after putting the sample into the container. On the paste there is a float which will sink in water but not in the paste. The top of the vertical glass stem attached to the float provides a target for a micrometer microscope by means of which the subsidence of the sample at the center of its top surface is measured. There are many details of procedure essential to the successful use of this technique which cannot be considered here. Certain cardinal points must be mentioned however: To obtain the correct rate of bleeding, the measurement must be that of a point on the surface far enough from the container wall as to be unaffected by the boundary friction; the radius of the sample should at least equal its depth. The total possible bleeding per unit height (bleeding capacity) must be determined on a sample shallow enough to permit complete settlement before the paste begins to congeal.

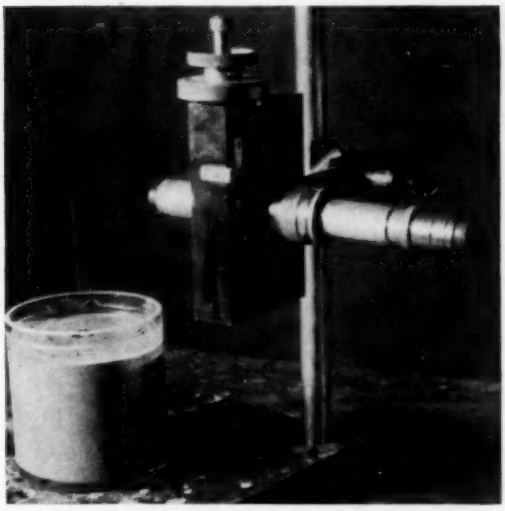

FIG. 1—GENERAL VIEW OF BLEEDING TEST SET-UP

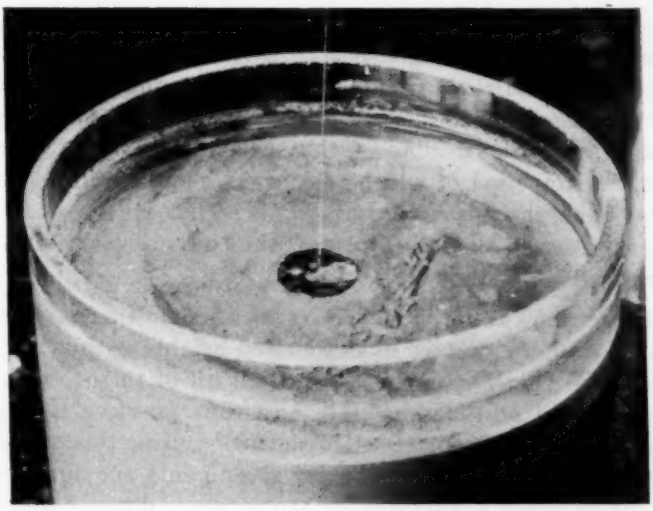

FIG. 2—CLOSE-UP OF FLOAT ON SURFACE OF PASTE

To determine the time interval before congealing begins, the sample must be so deep that mechanical consolidation cannot be completed within that interval. The temperature of the sample should be kept constant and not over about 70°F. In preparing paste samples, special care in mixing is required. The mixing must be vigorous, (more so than can be produced by hand mixing) but it must not so be violent as to cause an undue rise in temperature. It must be prolonged beyond the period of false set, usually 2 minutes. Methods based on the measurement of accumulated water give incorrect results because of the boundary friction effect. For a full explanation of these points, see the complete paper.

GENERAL ASPECTS OF BLEEDING

The whole process of bleeding is illustrated in Fig. 3. The vessel at the left may be considered to contain a sample of a plastic mix of paste, mortar, or concrete; they behave fundamentally alike. The similar figure at the right represents the same sample after the time interval of 140 minutes which, in this case, was required for the completion of the bleeding process. The intervening diagram traces the progress of changes that took place during this interval. The diagram is so constructed that the actual movements represented are multiplied by 10, thus clarifying certain otherwise obscure features.

The heavy line O′AD represents the movement of the surface of the sample. The surface is seen to subside at a steady rate for a definite period and then at a diminishing rate until it comes to rest. This is a typical bleeding curve, a straight line followed by a curve. Coincident with the decline of the surface, a layer of clear water appears, the top level of which remains at the original height of the sample. Thus, the process of bleeding is seen to be one of settlement or sedimentation of the solids through the body of water in which they are temporarily suspended.

The lighter lines below O′AD represent the simultaneous movements of points below the surface at designated heights within the sample. Except at the very bottom it is seen that all of these points at first move downward at the same rate and thus preserve their original spacing; there is no consolidation of the mass during this period except at the very bottom. Considering any given point, the duration of the period during which it moves at constant rate is directly proportional to its height above the bottom as shown by the straight part of the line O-A. After this period, the rate of settlement diminishes due to the progressive consolidation of the mix or, after about 75 minutes, to the effect of setting. Thus at any given time, as at 40 minutes for

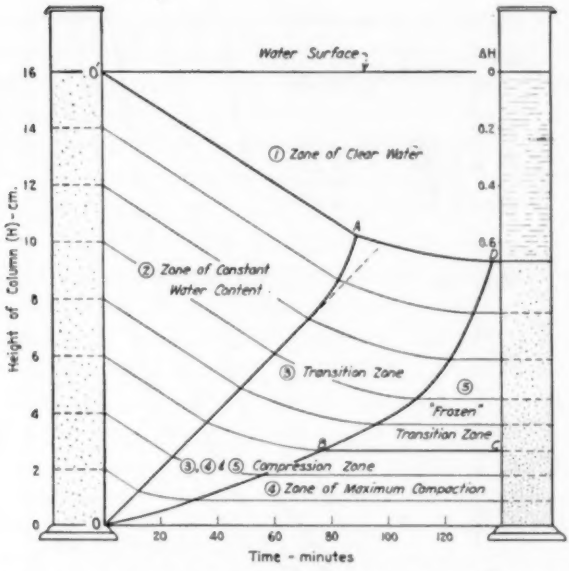

FIG. 3—AN ILLUSTRATION OF THE PROCESS OF BLEEDING BASED ON EXPERIMENTAL DATA

example, there exists a layer of free water at the top, a zone of constant water content, and at the bottom, a compression zone composed of completely and partly consolidated particles, the latter being in what is called the transition zone. The final stage represented at the right shows that after hardening, the sample may contain all of the zones, thus accounting for a density gradient from a maximum at the bottom to a minimum at the top.

The burden of this study was to find the laws governing the rate and amount of settlement or bleeding and the growth of the zones illustrated in this diagram, and then to apply these laws to practical problems of concrete work.

THE LAWS GOVERNING THE RATE OF BLEEDING

Bleeding is caused by the gravitational pull on the suspended particles, all of the particles, large and small, moving downward in unison due to their closely packed and coagulated condition. Opposing the downward movement is the frictional (viscous) resistance of the water flowing through the spaces between the particles. In the complete paper, it is shown that the law of Poiseuille correctly describes the factors governing the rate of bleeding of concrete. This law states that under a given hydraulic gradient and at a given temperature, the flow through capillary channels is directly proportional to the

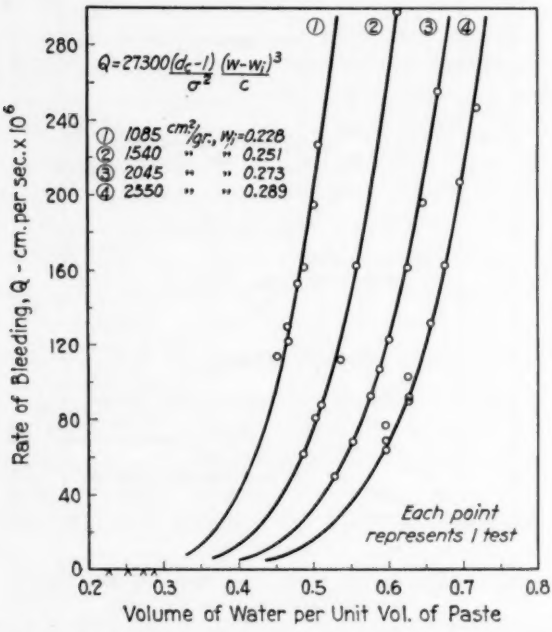

FIG. 4—COMPARISON BETWEEN OBSERVED AND CALCULATED BLEEDING RATES FOR ONE CEMENT GROUND TO FOUR DIFFERENT DEGREES OF FINENESS

fourth power of the capillary radius. Starting with this it was possible to derive an expression which gives the rate of bleeding in terms of the physical characteristics of the mix and of its components. The essential features of this expression are given below:

$$\text{Rate of Bleeding} = K \times \text{Fluidity of Water} \times \text{Force Causing Bleeding} \times \frac{(\text{Mobile Water Content})^3}{(\text{Surface Area Content})^2}$$

The constant K can here be considered to be a proportionality factor. Since the fluidity of the mixing water increases with its temperature, the rate of bleeding would increase with temperature. Experimentally this was found to be true within a limited temperature range; at temperatures above 80°, other effects reduced the bleeding rate.

The force causing bleeding is the suspended weight of the solids; the higher the specific gravity of the solids, and the greater the quantity of solids per unit volume of the mix, the greater the force.

The surface content is the amount of surface on all the solids in a unit volume of the mix. For a given quantity of solids, the higher their specific surface, the slower the bleeding rate.

The mobile water content is that part of the total water content that is free to flow during the bleeding process. A part of the water remains immobile with respect to the solids in the mix due to chemical or physical forces, or both. For any given material, the higher the mobile water content the higher the bleeding rate. The degree of agreement between measured rates of bleeding and the rates calculated by the above relationship is shown by Fig. 4.

IMMOBILE WATER FACTOR

The immobile water factor proved to be of great interest theoretically and of considerable practical importance. Due to differences in this factor, cements may vary in fineness as much as 500 sq. cm. per gram and still show the same rate of bleeding. The behavior of materials other than cement is influenced by this factor also. For example, a sample of pulverized limestone showed a considerably higher immobile water factor than one of pulverized silica, indicating that the silica must be ground finer than the limestone to show the same bleeding rate. It seems that important changes in the bleeding characteristics can be made by the use of soluble salts. With sodium chloride, for example, it was possible to reduce the bleeding rate of a given mix to less than one-half its original value. No comprehensive study of the immobile water factor was made; much remains to be learned regarding it.

CONTROL OF RATE OF BLEEDING

In approaching the question of control of bleeding, it is not to be assumed that bleeding should be eliminated entirely, for, as will become apparent, the measures required to reach that end are in some respects incompatible with other requirements for producing good concrete. Although the complete paper offers some guidance, in the main, the question as to what degree bleeding should be reduced is left open until more test data have been correlated with field experience. The discussion which follows is therefore restricted to a consideration of basic principles.

The equation discussed above shows that the two principal factors controlling the rate of bleeding are the proportion of mobile water and surface area content of the mix. If the practicable minimum water content still gives too much bleeding, then the amount of surface area per unit of water in the mix must be increased. This can be done successfully only by the use of materials of high specific surface, at least as high as that of normal portland cement. The reason for this is indicated partially in table 1 which shows the distribution of surface in several concrete mixes.

Even in the leanest mix, only 5 per cent of the surface is on the aggregate; the rest is on the cement. It is impossible to make sig-

TABLE 1—DISTRIBUTION OF SURFACE IN ORDINARY CONCRETE MIXES

Mix by Weight	Surface Areas—Sq. Cm. per Cu. Cm. of Concrete			
	No. 4¾ Gravel	No. 200-No. 4 Sand	Cement	Total
1—2	2	22	1365	1389
1—3	2	26	1040	1068
1—4	2	28	835	865
1—5	3	29	700	732
1—6	3	30	608	641

Specific Surface of Gravel — 6 sq. cm. per cc.
Specific Surface of Sand — 100 sq. cm. per cc.
Specific Surface of Cement—5800 sq. cm. per cc.

TABLE 2—EFFECT OF FINENESS OF THE CEMENT ON BLENDING RATE

Water Content of Paste		Bleeding (Subsidence) Rate, Cm. per Sec. x 10^6 Specific Surface of Cement, Sq. Cm. per Gm.			
w.	Gal. per Sack	1085	1540	2045	2550
0.45	2.96	103	39	17	9
0.50	3.58	210	80	38	20
0.55	4.39	—	150	73	40
0.60	5.37	—	270	133	75
0.65	6.68	—	—	223	128
0.70	8.36	—	—	—	213
0.75	10.70	—	—	—	—

nificant increases in the surface content of a mix by variations in the gradation of particles coarser than the No. 200 sieve. For this reason the control of rate of bleeding can be considered in terms of the composition of pastes alone.

The five principal ways of controlling the bleeding rate of a paste are as follows:

(1) Reducing the water per unit of cement by making the mix drier.

(2) Increasing the amount of cement per unit of water by making the mix richer.

(3) Increasing the fineness of the cement.

(4) Increasing the proportion of immobile water.

(5) Supplementing the cement with other fines of suitable specific surface.

The fourth item has already been discussed; of the others only the third and fifth will be considered here.

Effect of Fineness of the Cement. The effect of fineness of the cement, as indicated by its specific surface, is illustrated in Table 2 for one cement ground to four degrees of fineness.

It may be assumed that a bleeding rate of 100 x 10^{-6} cm. per second is the maximum that is desirable. On this basis the coarsest cement could be suitable only with less than 3 gal. water per sack of cement, a quantity which would not permit the use of aggregate. Finer grinding raises the maximum permissible water ratio and thus the aggregate

carrying capacity of a paste having this bleeding rate. With a specific surface of 1540 these figures show that about 4 gal. water per sack can be used; with a surface of 2045, about 5 gal.; with a surface of 2550, about 6 gal. If there are some fines of high specific surface in the aggregate these limits of water-cement ratio may be raised accord ingly. Also, if the immobile water factor were higher than it is in this particular case, the limits could be raised. It appears that most ordinary concrete made with well washed sand would show excessive bleeding, by this criterion.

Supplementing the Cement with Other Fines of Suitable Specific Surface. If it is desired to change the bleeding characteristics by supplementing the cement with another fine material, the quantity required to produce a desired change in bleeding can be calculated from the basic equations given in the paper. Table 3 shows how the specific surface of a material influences the quantity required for a given effect.

TABLE 3—PROPORTION OF SUPPLEMENTARY FINES REQUIRED TO GIVE DESIRED REDUCTION IN BLEEDING RATE

Proportion of Present Bleeding Rate Desired	Specific Surface Supplementary Fines	Ratio of Supplementary Fines to Cem. (Abs. Vol.)	Size or Kind of Particles
0.5	400	3.25	100–200 Mesh
0.5	4,000	0.550	15 microns
0.5	12,000	0.177	5 microns
0.5	60,000	0.033	1 micron
0.5	2×10^8	0.001	Colloids
0.5	5,000	0.430	Additional Cement

In this instance the computations are based on the requirement that the bleeding rate is to be reduced to one-half its original value. In the last column the kind of particles to be considered are indicated, and in the second column, the corresponding specific surface in square centimeters per cubic centimeter of material. The next to the last column shows the amount of additional material required per unit volume of cement in the original mix which may be considered to be one made with a cement having a specific surface of 5000 sq. cm. per cc. or about 1600 sq. cm. per gram. The last line shows that it would require an increase of 43 per cent of the same cement to reduce the bleeding rate to one-half its original value. To accomplish the same effect by the addition of 100-200-mesh material, the first line shows that a quantity equal to more than 3 times the volume of the cement would be required. This shows again the fultility of attempting to control bleeding with coarse material. Even the 15-micron particles shown in the next line would hardly be suitable, for a quantity equal to 55 per cent of the volume of the cement would be required with even these small particles.

Of particular interest is the indication of the effectiveness of a very small amount of colloidal material. Note that for such materials, which may be assumed to have a specific surface of the order of two million, only 1/10 of 1 per cent would be required to produce the same effect as 41 per cent of cement. This suggests a possible explanation for the apparent increase in the bleeding tendency of concrete noted in recent years; the more thorough washing which the aggregate now generally receives probably removes colloidal material which is very effective in the control of bleeding. Whether the gains due to this more thorough washing compensate for this effect on bleeding cannot be considered here.

It is necessary at this point to add a word of caution with respect to the interpretation of the foregoing discussion. It is important to bear in mind that the material most effective in the control of bleeding might not be the most desirable in all respects. In fact, experience has shown that materials of only moderately high specific surface are preferable because of the adverse effect of ultrafines on volume change and possibly on durability.

BLEEDING CAPACITY

The amount that a given mix can settle per unit of its original height is termed its *bleeding capacity*. The total amount that a given sample can settle is thus the product of its height and its bleeding capacity. The time required to reach complete settlement also increases with height so that there is a limit of height beyond which the settlement will not increase because the cement sets, arresting the process before its completion. The following discussion pertains to the period before setting begins and therefore to samples of limited height.

In the complete paper it is shown theoretically that the bleeding capacity of a cement paste depends upon the amount of water that it contains in excess of a certain base quantity. It works out that the bleeding capacity is approximately equal to one-third of the amount of water in excess of this base proportion. Fig. 5 gives the relationship between the bleeding capacity of a cement paste and its water content. The curve gives the theoretical relationship as computed from the basic equations given in the paper. The agreement between calculated and observed values is thus seen to be practically perfect except for the drier pastes. The explanation of the departure in this region is not essential to the present discussion. It is of interest that the base water-cement ratio shown on the diagram, 0.95 in this particular case, corresponds to the voids-cement ratio of this cement when compacted in a vacuum to the greatest possible density. Available space will

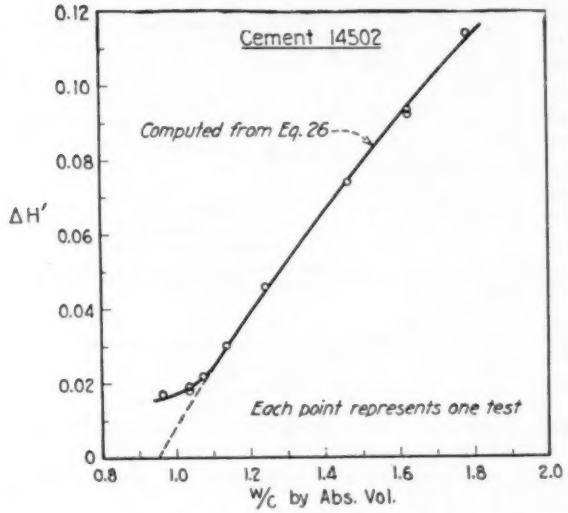

FIG. 5—RELATIONSHIP BETWEEN BLEEDING CAPACITY, ΔH', AND WATER-CEMENT RATIO OF NEAT CEMENT PASTE

not permit a complete explanation of this phenomenon; however, the fact that the maximum bleeding is approximately equal to one-third of the excess water is a consequence of the fact that the particles move only in the vertical direction during the process of settling.

Effect of Adding Aggregate. The effect of adding aggregate on the bleeding capacity of cement paste is shown in the upper diagram of Fig. 6. Each point represents the bleeding capacity of a sand-cement mortar expressed as bleeding capacity per unit of paste in the mortar. The heavy curve representing neat cements is reproduced from the diagram just discussed. It is seen that in the three richest mortars the bleeding capacities of the mortars per unit of paste were substantially the same as the bleeding capacities of the corresponding neat paste. From the standpoint of bleeding effects alone this is to be considered the ideal behavior, for it shows complete cooperation between the paste and the aggregate. In the leaner mortars, however, the bleeding capacity per unit of paste was very much less than that of the corresponding neat paste. This is direct evidence of premature bridging of the aggregate particles. It is such a lack of cooperation between paste and aggregate that gives rise to the detrimental effects of bleeding on the quality of the concrete; when the aggregate bridges

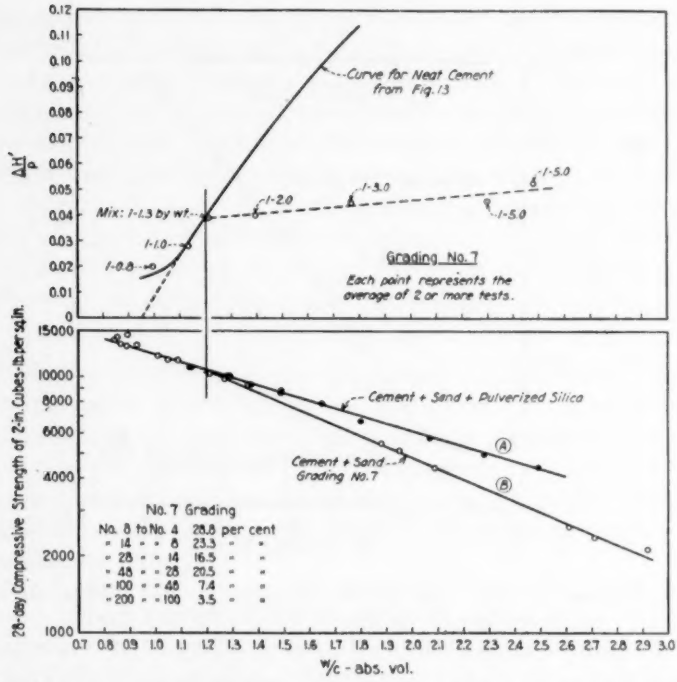
Curve for Neat Cement from Fig. 13
Mix: 1-1.3 by wt.
1-2.0
1-3.0
1-5.0
1-5.0
1-1.0
1-0.8
Grading No. 7
Each point represents the average of 2 or more tests.
Cement + Sand + Pulverized Silica
Cement + Sand Grading No. 7
(A)
(B)
No. 7 Grading
No. 8 to No. 4 28.8 per cent
" 14 " " 8 23.3 " "
" 28 " " 14 16.5 " "
" 48 " " 28 20.5 " "
" 100 " " 48 7.4 " "
" 200 " " 100 3.5 " "
28-day Compressive Strength of 2-in. Cubes-lb. per sq.in.
W/c - abs. vol.

FIG. 6—EFFECT OF PREMATURE BRIDGING OF THE AGGREGATE ON COMPRESSIVE STRENGTH OF SAND-CEMENT MORTARS

before the paste becomes consolidated, the paste continues to settle and separates from the undersides of the aggregate particles.

The effect of this separation on strength is shown in the lower diagram of Fig. 6. The solid points represent ideal mixes whose bleeding capacities per unit of paste are the same as that of the neat paste in all instances. These ideal mixes were obtained by supplementing the cement with pulverized silica of about the same fineness of the cement, as required. The open circles represent the same series of mixes represented in the upper diagram; no supplementary fines were used. The open circles follow the same straight line as the closed ones in the range of very rich mortars; that is, the rich plain mortars show the same water-cement ratio strength relationship as the ideal mortars, but in the leaner mixes the strengths of the plain mixes were in every case less than that of the corresponding mixes containing pulverized silica. It is very significant that the water-cement ratio at which premature bridging is manifested in the upper diagram is precisely that at which the two strength curves of the lower diagram diverge.

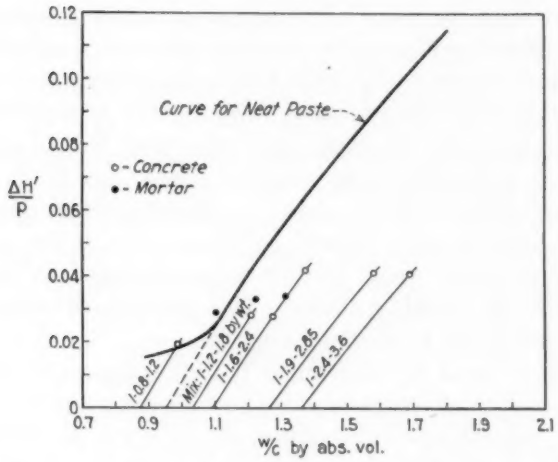

FIG. 7—BLEEDING CAPACITIES OF MORTARS AND CONCRETES PER UNIT OF PASTE IN THE MIX COMPARED WITH THE BLEEDING CAPACITIES OF NEAT PASTES OF THE SAME WATER-CEMENT RATIO

Other data of this nature are shown in Fig. 7, in which the points represent concrete mixes and mortars of the same kinds as in the concrete mixes. Here, the premature bridging is manifested in leaner mixes than those shown in Fig. 6, 1-1 by weight as compared to 1-1⅓. This denotes somewhat inferior grading and shows that the degree of premature bridging can to some extent be controlled through the gradation of the aggregate.

The curves for the respective concrete mixes were calculated from the fundamental relationship between bleeding capacity and excess water content. It works out that the base water content in each mix is that which would give a slump of 0.7 in. This provides a very simple rule by which to determine whether there will be premature bridging in a given mix; namely, *if the water-cement ratio of a given concrete mix at a slump of 0.7 in. is greater than the base water ratio of a neat paste of the same kind as that in the concrete, premature bridging will occur. On the other hand, if the base water ratio is equal to or less than that of the neat paste, there will be no premature bridging.*

It should be noted that those mixes which indicate no premature bridging were considerably richer than any commonly used. The indications are that with a cement of ordinary bleeding characteristics, even with the best gradation of the particles coarser than the 200-mesh sieve, it is not likely that mixes leaner than about 1-1½-X could be kept free from premature bridging. To prevent premature bridging

in the leaner range, it seems to be necessary to change the characteristics of the paste itself so as to raise its base water-cement ratio and shift the whole curve to the right. Some phases of the control of the characteristics of the cement paste have already been discussed.

Finally it must be re-emphasized that it is by no means to be assumed that mixes ideal with respect to bleeding effects are likewise the most desirable in other ways. It might be necessary to accept some imperfections with respect to bleeding in order to safeguard other properties that would be adversely influenced by the measures taken to eliminate bleeding effects. This pertains especially to shrinkage and swelling due to moisture changes.

In closing, it must be made clear that this discussion gives a very incomplete picture of the phenomenon of bleeding and its significance. To indicate something of the nature of the material which has been omitted or only treated sketchily, the following subjects treated in the complete paper are mentioned:

(1) *The derivation of the fundamental equations.* Although the derivations are for the most part relatively simple substitutions in a fundamental formula, it is necessary to understand them in order to comprehend fully the significance of the final expressions obtained.

(2) *The five zones of compaction.* From a knowledge of the rate at which the various zones of compaction within the mass of concrete develop, the proportion of the total mass that is compacted by the bleeding process can be estimated.

(3) *The effect of added aggregate on the bleeding rate of the cement paste.* This study reveals much that is fundamental to the control of the basic texture of concrete itself.

(4) *Adsorption and the immobile water factor.* This is perhaps one of the most important developments of the paper from the academic viewpoint. It opens up a new field of study as to the influence of salts on the bleeding characteristics and possibly also plastic characteristics of pastes in concrete.

(5) *The relation of bleeding capacity to slump.* There seems to be a fundamental relation of slump and the amount that a mix can settle.

(6) *Duration of the period of constant rate and the rate of rise of the compression zone.*

(7) *Boundary friction.* This is an extremely important phase of the phenomenon completely ignored in the foregoing discussion. Under some circumstances the effect of friction against the molds or fixed objects within the forms predominate in controlling the behavior of the mix.

(8) *The effect of temperature.*

(9) *The effect of entrained air.* It is shown that air bubbles, produced and stabilized during the mixing period by certain foaming agents, are very effective in reducing the rate and amount of bleeding.

(10) *Channeling.* In extreme cases of bleeding, water flows through localized channels and develops "sand boils" at the surface. A possible explanation of this phenomenon is given.

(11) *Bond between concrete and steel.* In the present discussion the steps required to prevent damage to the structure of the mix itself were considered. Other factors must be taken into consideration when it is necessary to minimize the separation between concrete and embedded reinforcement.

(12) *Scaling and delayed finishing.* Certain types of scaling are explained in terms of bleeding phenomena and the fundamental soundness of the delayed finishing is demonstrated.

(13) *The technique of measuring bleeding.* There are many factors to be carefully controlled in order to obtain reliable data.

* * *

Although it is believed that the study in its present stage has thrown considerable light on a good many phases of bleeding and on its significance, it probably has opened up even more questions that are to be answered by future investigations. Of particular interest is the relationship between premature bridging and particle interference. Also it is important to study further the properties of mixes that are ideal from a bleeding standpoint to learn to what extent they can be used without sacrificing other necessary properties. Again, it is highly desirable to apply the principles disclosed from a bleeding study to a study of plasticity and mix design in general. Finally, an extensive study of the immobile water factor seems to be in order.

Discussion, to close in the September Supplement, should reach the A. C. I. Secretary by July 20.

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(Supplement)

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Discussion of a paper by T. C. Powers:

The Bleeding of Portland Cement Paste, Mortar and Concrete*

BY WINSLOW H. HERSCHEL, M. O. WITHEY, J. W. KENNEDY AND T. C. POWERS (AT THE CONVENTION) AND THADDEUS MERRIMAN AND AUTHOR'S CLOSURE

CONVENTION DISCUSSION

Winslow H. Herschel†—It is very interesting to see sedimentation discussed on the basis of Poiseuille's Law instead of the usual Stokes Law, but there are two points in regard to the derivation of the equation for the rate of bleeding, upon which there seems to be need—possibly it is given in the complete paper—for elucidation. One is the introduction of specific surface. It would seem as though the effective specific surface would be materially changed the minute that cement and water are mixed, and therefore that the original specific surface would not be very effective in producing a certain rate of bleeding. The other point is that although friction against the sides of the vessel was mentioned and although it was explained that there were compaction zones, thus implying that the particles certainly were in contact and that there was relative motion of one particle in regard to another, there appears to be no factor in the equation which depends upon the friction between the different particles. It is not clear just why it is permissible to neglect the friction between one particle and another.

T. C. Powers—The question regarding specific surface is pertinent, and I think it is answered in the complete paper. But I hesitate to take the time of you who have not seen the complete paper and the

*JOURNAL, Amer. Concrete Inst., June 1939; *Proceedings* Vol. 35, p. 465.
†National Bureau of Standards, Washington, D. C.

derivations to go into a detailed explanation of the point that Dr. Herschel has raised. I might say that the best defense I have of the assumptions made is the correlation between the observed and calculated values.

There was no evidence during the period of constant rate (remember that these relationships deal with the period of constant rate) that there is friction between the particles. In fact, within zone 2, to which the rate equation applies, the particles do not move with respect to each other; hence there is no interparticle friction. The evidence is that the particles, although coagulated, are nevertheless separated by a small space and remain so throughout the period during which they settle at constant rate. I do not want to sidestep your question, but I hesitate to go into a complete discussion because of the time it would require.

*M. O. Withey** — I may be entirely out of order in asking this question, but it seems to me that the evidence which has been presented here and that others have accumulated along this line, indicates that this whole phenomenon is one of settlement or sedimentation. Why do we preserve this term bleeding?

Chairman Pearson—That is what it looked like when it started, Professor Withey, and when you get an impression like that, there seems to be a multitude of supporting circumstances. I agree with you it is misleading.

Professor Withey—Can't we all discard it then?

A Member—Our English friends call it weeping.

J. W. Kennedy†—I am thinking of what Mr. Earley stated day before yesterday, of the trend toward the highly theoretical and academic aspects of many of our research problems. Mr. Powers has presented a wonderful paper, but I wonder if he has warned those of us in the audience who pride ourselves on being somewhat practically inclined against accepting his data too literally, especially since there are so many loose ends involved? Now I look at the curves and I see the wonderful effect of certain jells in correcting these phenomena that he has observed, and the effect of increased fineness of the cement and other possible things we can do to improve what we call bleeding. Now we will go back to the job and we will start applying one of those things to the utter disregard of what might happen. Doing so, then we may cause more damage to the concrete than if we had tolerated some bleeding in the first place. I think we cannot be warned too much against a literal interpretation of the data presented, because I

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know Mr. Powers would be the first one to admit there is a lot to be learned yet on this question of sedimentation.

Mr. Powers—I think Mr. Kennedy is right. For example, it is indicated in the paper that a little colloidal clay is very effective for reducing bleeding rate, but I would hesitate to add colloidal clay to control bleeding because I believe it could be detrimental to other properties. I do think, however, that we should study the different methods of control suggested and as soon as we are certain that they are foolproof, we should use them.

THADDEUS MERRIMAN*

By emphasizing the great differences which manifest themselves in the behavior and bleeding characteristics of various "standard" portland cements the author has served a most useful purpose. Representing a vast labor, his investigation covers a wide field and his ingenious solutions of many complex and interrelated variables commands attention.

While the paper presents the chemical analyses of some of the cements used in the investigation a number of them are omitted and, with respect to the concretes and mortars, there is no statement as to what cement was used. More importantly, however, the history of none of the cements is given. At what temperatures were they calcined? Was the clinker cooled with water? Was it stored outdoors before grinding and, if so, for how long? From what plant did the cement come? These points and others press for answer because the quality and characteristics of the cement vitally depend on them. No complete and convincing study of bleeding, or any other subject related to cement, can be made without a full consideration of all the variables which enter the problem. Not the least of these are the heritages impressed by the procedures of manufacture. The very same raw material processed at two plants may, and probably will, produce two cements differing widely in their qualities and characteristics. Not until these differences are recognized and their effects studied and evaluated will progress in the Science of cement become possible.

Referring to the ignition losses presented in Table 1 it is seen that they range from 0.66 to 2.22 per cent and that 11 of the 16 cements showed a loss of over one per cent. The free lime ranged from 0 to 1.87 per cent and 7 of the 16 showed values in excess of 0.70 per cent. Nearly all had their SO content above normal, only K-17 was down to 1.27 and, it is worthy of note that this cement not only had the

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lowest free lime (0.00) but also the lowest ignition loss (0.66). With respect to composition this cement was probably the best in the group. In fact, it is outstanding in the difference shown in Fig. 5F where its curve is steeper than that of any other of its competitors, though it must be noted that it was very coarsely ground. But the reason for this is not stated.

In the absence of the histories of the cements experimented with the correlations obtained are as satisfactory as any that could be developed and the writer heartily joins the author in the caution, several times expressed, that his results were obtained in the laboratory and may not represent the behavior of any cement when used in concrete construction.

The author tentatively explains the observed "variations in w_i"* * * on the basis of the theory of solvation." This phenomenon may act its part but, to one who has worked with many cements and observed their behaviors, it seems much more probable that the major role is played by the rate at which a flock or gel is produced and on the amount thereof. In the case of a cement containing a substantial proportion of underburned material fllock will be generated much more rapidly than in one which has been completely calcined at the proper temperature and during an optimum time interval. On the other hand, a completely processed cement, because of its slower reaction rate, will hydrate and form floc much less readily and so will have a higher bleeding rate. As between two such cements of the same composition the latter will make the better and more durable concrete.

The author states that calcium chloride "appears to very effective" in increasing the value of w_i "though possibly only through its acceleration of setting." But it should be remembered that the principal effect of this salt is its action in converting the alkali hydroxides into the neutral chlorides and thus removing them from the picture. The overall effect is thus to cause a marked change in the *pH* of the solution in which the hydration processes occur. By lowering the *pH* the hydrolosis of the cement proceeds in a different environment and at a different rate.

While it is undoubtedly true that "a small amount of clay or silt in the aggregate reduces the bleeding rate markedly" it is of first importance to know whether this clay reacts injuriously with the cement. Some clays in "small amount" will utterly ruin concrete and care must always be exercised in accepting any aggregate which has not been thoroughly washed.

While this paper largely confines itself to a discussion of the phe-

nomena of "bleeding" it also directs attention to three other important items as follows:

(a) The danger inherent to the barbarous practice of finishing concrete surfaces by sprinkling on dry neat cement.

(b) The value of delaying the final finish as long as possible thus permitting a maximum of settlement and bleeding to occur. Bleeding of itself is not as injurious as is sometimes pictured. Going one step further the paper might well have discussed the desirability of overfilling the forms and removing the surplus just before the final finishing operation is begun.

(c) Most importantly of all the paper seems to intimate that the commonly accepted methods of designing concrete mixes are badly in need of revision.

With respect to this latter point it should be said that no concrete is better than the cement it contains nor is any concrete better than the mortar (the sand and the cement together) with which it is made. Good mortar will rarely contain less than one part of cement to 1.75 parts of sand by volume. Coarse aggregate is then to be added to this good mortar in such amount and of such sizes as may be suitable for the operation in hand. But really workable and watertight concrete can be made only when the cement content is from 7 to 8 bags of cement per cu. yd. In every such mix, however, the quality of the cement is of prime importance and only those cements which have been so processed as to contain no more than small fractions of underburned material can be employed without the occurrence of excessive crazing, checking and cracking.

From the viewpoint of actual experience in the placing of concrete the following observations are pertinent. They are presented here because the author's discussion of bleeding will be certain to create in many minds the impression (a) that bleeding is a harmful phenomenon and (b) that the best cement will bleed the least. Neither of these conclusions would be warranted because the facts are to the contrary. Let it also be noted that the author did not so express them.

Every thoroughly burned and carefully processed cement will bleed when mixed in concrete. Fine grinding tends to reduce bleeding but bleeding is a direct evidence of quality and the better a cement, all other conditions being the same, the more it will bleed. This is true because, in the best cement, there is a minimum of dead material and a minimum of hydration occurs during the period of mixing and placing. On the other hand, a poorly burned and carelessly handled

cement will fluff up to such an extent that water can flow upward through the resulting gelatinous mass hardly at all, with the result that there is relatively little bleeding. Is such a cement for that reason alone to be considered as best? In one case of which the writer has been told a particular cement with five bags in the mix bled less than another when six bags were used. This behavior led the engineers to believe that the five-bag cement was the better whereas it was actually much the poorer. That job did not get its money's worth even if it did save one bag of cement for each cubic yard of concrete!

Now if the best cement is the worst bleeder what is the engineer to do about it? The answer is threefold. In the first place if he considers the bleeding excessive he will redesign his mix by adding more of the same best cement until the bleeding is checked or ceases entirely; only then will the mix be properly designed. In the second place he will observe, even if his mix does bleed, that the accumulation of laitance will be so very small as to be practically negligible. In the third place he will know that by whatever amount his concrete settles and the water rises to the surface the denser and the better it will be. A concrete in which the cement fluffs up so as to prevent settlement is always of poor quality.

It thus appears that bleeding can be controlled; second, that with a well made cement it is not necessarily harmful and, third, that it may improve the concrete. But, of course, as between these alternatives, there is, as in all physical operations, a procedure which will give the best overall result in the completed concrete. Stated broadly it is (1) use the best cement obtainable in such quantity that the bleeding will be within reasonable bounds; (2) use such quantity of water as will produce a maximum density in the finished concrete. This procedure, it will be observed, conflicts with the so-called law of the "cement water ratio" which prescribes the use of a minimum of water for a workable mix. In the case of the best cement which is here considered, however, it will not be injured by exposure to a quantity of water which would practically ruin the average product. So it is that more water can be properly used with a good cement than with a poor one and that the phenomenon of "bleeding," considered by itself alone, is not a sound criterion of quality. The employ of more water than a so-called optimum is, in many cases and perhaps in all, highly desirable from the angle of the overall quality of completed concrete.

In order to produce durable and permanent concrete the conditions at all stages of the manufacture of the cement and its storage before use must be right. Then, next, the concrete mixing and placement

must be done with skill and with judgment. Finally, the concrete may not be left as an orphan on the doorstep but must be wet cured almost from the moment of its placement and continuously thereafter for 14 mays or more. These three things are jointly necessary to develop the hard, dense and rocklike amorphous constituent which denotes good concrete and is the symbol of permanence and durability.

AUTHOR'S CLOSURE

With respect to Mr. Merriman's discussion of the differences between cements, the author can only refer to the many pages of discussion in the complete text which show that the author, like Mr. Merriman, believes that the chemical and physical properties of a cement, as influenced by its composition and the method of manufacture, should have an important bearing on its bleeding characteristics. But it cannot be agreed that the differences in bleeding characteristics are due primarily to the rate of formation of reaction products, for reasons set forth in the complete paper. The theory toward which Mr. Merriman inclines was studied rather carefully before the work was published. It had to be rejected because it was not compatible with the experimental results. The study reported did not extend to the point of determining which chemical and physical properties are responsible for the differences in question; that is a proper subject for further study. When such a study is made, it will be necessary to obtain as complete information as is available about each cement used, as Mr. Merriman points out.

Mr. Merriman expresses the belief that "the best cement is the worst bleeder." Selecting one of the cements as having what he considers to be the most favorable characteristics of any of those reported, he refers to the relatively great steepness of its bleeding curve as support for this belief. However, the fact is that this particular cement, which was one of an experimental series, was not intrinsically a bad bleeder. The steepness of its bleeding curve was due to its low specific surface, which of course has nothing to do with the characteristics of the original clinker. In fact, the steepness of any bleeding curve depends almost entirely on specific surface, and cements do not exhibit individuality with respect to the effect of this factor. Cements do manifest their individualities through differences in w_i; a relatively low value of w_i at a given specific surface indicates an intrinsically bad bleeder. The cement in question did not have a low w_i value. This example, therefore, cannot be taken to support Mr. Merriman's belief.

In general, the author believes that Mr. Merriman's contention that the best cement is the worst bleeder lacks satisfactory proof and

should not be accepted until supporting data are available. On the other hand, Mr. Merriman correctly emphasizes the fact that the author does *not* state that the best cement will bleed the least. As a matter of fact the author knows nothing definite of any relationship between bleeding characteristics (other than those controlled by fineness) and other properties, and has seen no data that will warrant anything more positive than a guess as to what such relationships might be.

Mr. Merriman mentions that the author's discussion of bleeding is certain to lead to the belief that bleeding is a harmful phenomenon. It should be pointed out that the author (in the complete paper) does state definitely that the effects of bleeding are harmful under certain specified conditions. It may be added that those conditions are believed to prevail widely and that much concrete suffers to some degree from the effects of bleeding, or, more accurately, from the effect of premature bridging. But in each case where correction seems desirable, it should always be ascertained whether the proposed cure is preferable to the malady.

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Tests of Concrete Curing Materials*

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SYNOPSIS

The tests reported in this paper were made with the twofold idea of obtaining comparative data on the effectiveness of various curing materials and methods now in common use and of developing a standard laboratory test procedure for use in specifications. The curing materials which were investigated included, in addition to burlap, calcium chloride, used both as a surface application and as an admixture; sodium silicate; six waterproof papers; a special curing blanket consisting of two layers of burlap with a jute bat between; an asphalt emulsion; an asphalt cutback; a straw-colored lacquer-like liquid; and a rubber (latex) emulsion. The value of cotton mats for curing concrete has been demonstrated conclusively both in the laboratory and in the field. For this reason cotton mats were not included in this study.

The effectiveness of each curing method was measured both in terms of relative moisture loss and relative strength, using test specimens of 1:2 mortar, 6½ in. wide, 12 in. long, 2 in. deep. The curing material was applied to the top or molded surface of the specimen and sealed around the edges in such a manner that moisture could escape only through the curing medium itself. The rate of mositure loss was measured for each method by determining the loss in weight at various intervals during the exposure period.

The results secured in this investigation seem to warrant the following general conclusions:

(a) As regards burlap used alone: (1) The effectiveness of burlap curing is increased by lengthening the duration of application. (2) The effectiveness of burlap curing is decreased by increasing the time elapsing between the placing of the concrete and the application of the burlap. (3) Burlap curing is not as effective when sprinkled intermittently as when kept continuously saturated.

*Presented by Mr. Jackson at the 35th Annual Convention, American Concrete Institute, New York, March 1-3, 1939.

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(b) As regards surface sealing materials: (1) The effectiveness of such materials as waterproof paper and liquid curing materials applied with a spray is materially increased when preceded by a 24-hour application of wet burlap. (2) The effectiveness of such membrane coverings as liquid curing materials A and B is materially improved by applying the covering 3 hours after molding as compared to an application made 1 hour after molding.

INTRODUCTION

Numerous methods and procedures used for curing concrete pavements are included in the scope of the investigation herein reported. However, the investigation did not include the use of cotton mats which have proven highly effective as a curing medium, not only on account of their ability to retain moisture over a considerable period but also because they protect the concrete from large fluctuations in temperature at early ages when its ability to resist temperature stresses is low. The reason for the omission of cotton mats was twofold. First, the investigation did not involve any study of thermal insulation but was for the purpose of determining the ability of various curing agents to retain moisture and thereby promote the development of strength. In the burlap curing that was used as a basis for the comparison of the other methods, the burlap was kept in a saturated condition at all times and therefore, insofar as moisture loss is concerned, the results were the same as would have been obtained with saturated cotton mats. Second, the technical and practical advantages of cotton mats have been demonstrated conclusively by previous investigations, both in the laboratory and in the field.

Thorough and complete curing has always been recognized as one of the most important single factors involved in the construction of a concrete pavement. The importance of delaying moisture loss until the concrete has attained sufficient strength to furnish high resistance to the shrinkage stresses resulting from drying is self-evident. For this reason, provisions for curing form a very important part of every concrete pavement specification.

For many years concrete pavements were cured almost entirely by means of a thorough and continuous application of water for periods up to 10 days after placing. Curing began by covering the concrete with wet burlap applied just as soon after finishing as possible. This was kept continuously wet until the following day when it was replaced by a covering of earth or straw kept continuously wet for periods of from 7 to 10 days. It has always been pretty generally agreed that, theoretically at least, such a method is ideal. However, it requires continuous wetting over a considerable period, a procedure which is

not only expensive but requires constant and efficient supervision to insure full compliance.

So long as only small amounts of pavement were involved and daily yardages were limited, it was possible to enforce such continuous curing provisions without great difficulty. However, as methods of construction became more efficient, and daily yardages increased, the cost of curing by water as well as the difficulty of enforcing the requirements mounted rapidly. As was bound to happen, this condition has resulted within the last several years in the introduction of numerous substitute methods of curing, designed to accomplish the same purpose without the use of water. Most of these, such as various grades of waterproof paper coverings, sodium silicate, liquid bituminous products, rubber emulsion, etc., depend entirely for their efficiency on their ability to retain water within the concrete. They are surface sealing materials and their use is justified on the theory that adequate curing can be accomplished by retaining the contained water.

Many attempts have been made from time to time to study the effectiveness of different methods of curing concrete through the construction of experimental roads, curing different sections by different methods. Such procedure would seem to be a very logical method of ascertaining the comparative value of different curing materials. Actually, however, the impossiblity of controlling other variables which may affect the result, particularly weather conditions, make it of distinctly questionable value. There are many problems regarding concrete pavement construction which may be studied with profit through the construction of experimental roads. However, in the authors' opinion, curing is not one of them. It is believed such comparisons should be made only in the laboratory under closely controlled temperature and humidity conditions, using a test procedure which will permit direct comparisons of the efficiency of different materials. Having determined by means of a series of tests of this type, the degree of compliance which may reasonably be expected, suitable requirements could be written into standard specifications and the test procedure used as a standard routine laboratory method of evaluating the various materials and processes which are offered for use.

PURPOSE AND SCOPE OF TESTS

The tests reported herein were made with the twofold idea of obtaining comparative data on the effectiveness of various curing materials and methods now in common use and of developing a standard laboratory test procedure for use in specifications. The procedure followed has been made available to Committee C-9 of the A. S. T. M. in

developing a tentative method for testing curing agents. The curing materials which were investigated included, in addition to burlap, calcium chloride, used both as a surface application and as an admixture; sodium silicate; six waterproof papers; a special curing blanket consisting of two layers of burlap with a jute bat between; an asphalt emulsion; an asphalt cutback; a straw-colored lacquer-like liquid; and a rubber (latex) emulsion. The last four materials were proprietary liquid curing compounds applied in the form of a spray. A brief description of each of the materials investigated is given in Table 1.

TABLE 1—DESCRIPTION OF CURING MATERIALS

Type	Description
Burlap	Weight, 9 ounces per square yard.
Paper A	Two layers of paper cemented together with bitumen and reinforced with sisal fibers.
B	Two layers of paper, reinforced in both directions at about one half-inch intervals and cemented together with bitumen, bitumen applied to one layer only.
C	Same as paper B, except bitumen applied to both layers of paper.
D	Single layer of unreinforced paper, treated with a white emulsion.
E	Same as paper D, except treated with a brown emulsion.
F	Same as paper D, except treated with a brown-white emulsion.
Sodium Silicate	Commercial grade as used for curing concrete. Applied with a brush.
Calcium Chloride	Standard commercial product (flake) as used for curing concrete.
Curing Blanket	Consists of two layers of burlap with jute bat between. Weight 22 ounces per square yard.
Liquid Curing Material A	Special asphalt emulsion used for curing concrete. Applied with a spray gun.
Liquid Curing Material B	Special asphalt cut-back used for curing concrete. Applied with a spray gun.
Liquid Curing Material C	A straw colored lacquer-like liquid. Applied with a spray gun.
Liquid Curing Material D	A rubber emulsion (latex) applied with a spray gun.

NOTE—The liquid curing materials are all proprietary compounds, the exact composition of which was not determined.

Several of the surface sealing mateials were used both with and without a preliminary 24-hour application of wet burlap. In addition, the time elapsing between the molding of the specimen and the application of the curing material was varied. The comparative effect of burlap curing for 1 day, 2 days, and 3 days without subsequent curing was also investigated. In all, 38 different curing procedures involving 14 materials were studied, the results being compared with the results obtained with specimens cured continuously with wet burlap in sealed containers (ideal curing) as well as with specimens exposed to the air without protection of any kind (no curing). To provide ideal curing the specimens were first covered with two layers of wet burlap. Over this was placed a metal cover which was sealed around the edges to prevent any loss of moisture.

The results of five series of tests, four after 7 days' exposure under the temperature and humidity conditions described below, and one after 28 days' exposure are reported. A brief description of each procedure, including the type of curing material involved, whether used with or without an initial application of burlap, the time of application

TABLE 2—DESCRIPTION OF CURING PROCEDURES

Method No.	Curing Procedure	Used in Series				
		A	B	C	D	E
1a	Wet burlap, sealed with metal cover continuously	x	x	x	x	x
2	No curing	x	x	–	x	x
3a	Wet burlap for 1 day, applied immediately after molding	x	–	–	x	x
4a	Wet burlap for 2 days, applied immediately after molding	x	x	–	x	x
4b	Wet burlap for 2 days, applied 1 hour after molding	–	x	–	–	–
4c	Wet burlap for 2 days, applied 3 hours after molding	–	x	–	–	x
4c-1	Wet burlap for 2 days, applied 3 hours after molding	–	x	–	–	–
4c-2	Wet burlap for 2 days, applied 3 hours after molding	–	x	–	–	–
5a	Wet burlap for 3 days, applied immediately after molding	x	x	–	x	x
5b	Wet burlap for 3 days, applied 1 hour after molding	–	x	–	–	–
5c	Wet burlap for 3 days, applied 3 hours after molding	–	x	–	–	x
5c-1	Wet burlap for 3 days, applied 3 hours after molding	–	x	–	–	–
5c-2	Wet burlap for 3 days, applied 3 hours after molding	–	x	–	–	–

NOTE—For methods 4c-1 and 5c-1 the burlap was sprinkled intermittently in such manner as to keep it continuously wet.
For methods 4c-2 and 5c-2 the burlap was sprinkled intermittently and allowed to become dry between wettings.

6a	Wet burlap for 1 day, applied immediately after molding followed by paper A for 6 days	x	x	–	x	x
6b	Wet burlap for 1 day, applied 1 hour after molding, followed by paper A for 6 days	–	x	–	–	–
6c	Wet burlap for 1 day, applied 3 hours after molding, followed by paper A for 6 days	–	x	x	–	x
7a	Wet burlap for 1 day, applied immediately after molding, followed by sodium silicate	x	–	–	x	x
7c	Wet burlap for 1 day, applied 3 hours after molding, followed by sodium silicate	–	–	–	–	x
8a	Wet burlap for 1 day, applied immediately after molding, followed by calcium chloride (surface application)	x	–	–	x	x
8c	Wet burlap for 1 day, applied 3 hours after molding, followed by calcium chloride (surface application)	–	–	–	–	x
9a	Wet burlap for 1 day, applied immediately after molding, followed by liquid curing material A	x	–	–	x	x
9c	Wet burlap for 1 day, applied 3 hours after molding, followed by liquid curing material A	–	–	x	–	x
10c	Wet burlap for 1 day, applied 3 hours after molding, followed by liquid curing material B	–	–	x	–	–
11a	Wet burlap for 1 day, applied immediately after molding, calcium chloride admixture	x	–	–	x	x
11c	Wet burlap for 1 day, applied 3 hours after molding, calcium chloride admixture	–	–	–	–	x
12c	Curing blanket, for 3 days, applied 3 hours after molding	x	–	–	x	x
13b	Waterproof paper A, for 7 days, applied 1 hour after molding	–	x	–	–	–
13c	Waterproof paper A, for 7 days, applied 3 hours after molding	x	x	x	x	x
14c	Waterproof paper B for 7 days, applied 3 hours after molding	x	–	–	x	x
15c	Waterproof paper C for 7 days, applied 3 hours after molding	x	–	–	x	x
16c	Waterproof paper D, for 7 days, applied 3 hours after molding	x	–	–	x	x
17c	Waterproof paper E, for 7 days, applied 3 hours after molding	x	–	–	x	x
18c	Waterproof paper F, for 7 days, applied 3 hours after molding	x	–	–	x	x
19b	Liquid curing material A, applied 1 hour after molding	–	x	–	–	–
19c	Liquid curing material A, applied 3 hours after molding	x	x	x	x	x
20b	Liquid curing material B, applied 1 hour after molding	–	x	–	–	–
20c	Liquid curing material B, applied 3 hours after molding	x	x	x	x	x
21b	Liquid curing material C, applied 1 hour after molding	–	x	–	–	–
21c	Liquid curing material C, applied 3 hours after molding	x	x	–	x	x
22c	Liquid curing material D, applied 3 hours after molding	x	–	–	x	x

and the duration of application are shown in Table 2. This table also indicates the series in which each procedure was used. In series A to D inclusive, the specimens were exposed for 7 days in an atmosphere maintained at 100° F. ± 2° F. with a relative humidity of 32 per cent ± 2 per cent using for this purpose a specially designed curing cabinet in which the temperature and humidity were controlled automatically within the limits indicated. In series E, the specimens were exposed for 28 days. Each result reported in series A to D, inclusive, with the

exceptions noted in series B, is the average of either five or six individual determinations made on different days. The results of the 28-day tests series E, are the average of from two to five specimens, as noted.

In series A, 19 methods in addition to the standard or ideal and the no-cure, were investigated. These included burlap for 1, 2, and 3 days; paper A and liquid curing material A with and without burlap; the other papers and liquid curing materials without burlap, that is, as recommended by the manufacturers; calcium chloride, both as a surface application and as an admixture; sodium silicate and the curing blanket. It will be noted that in this series, surface sealing materials when used without burlap were applied 3 hours after molding. This would represent about the maximum time which might be required in the field. Burlap in this series, however, was applied immediately after molding.

Series D was a duplication of series A, run several months later. In series B the effect of varying the time of application of the curing agent was investigated. In the case of burlap, the effect of delaying the application 1 hour and 3 hours is shown as well as the effect of continuous and intermittent sprinkling. The relative effect of applying paper A and liquid curing materials A, B, and C, 1 hour after molding, as well as 3 hours, is also shown. Series C was run in order to obtain additional data on the effect of using waterproof paper and liquid curing material with a preliminary 24-hour application of wet burlap for comparison with the usual method which does not require burlap. Series E gives the results of a series of tests after 28 days' exposure, using, in general, the same methods as used in series A and D in which the specimens were exposed 7 days.

PROCEDURE

The effectiveness of each curing method was measured both in terms of relative moisture loss and relative strength, using test specimens of 1:2 mortar, 6½ in. wide by 12 in. long by 2 in. deep. The curing material was applied to the top or molded surface of the specimen and sealed around the edges in such a manner that moisture could escape only through the curing medium itself. The rate of moisture loss was measured for each method by determining the loss in weight at various intervals during the exposure period.

The specimens were molded in watertight sheet metal pans, the bottoms of which were reinforced with angle sections for a stiffening effect. This was done because it was necessary in many cases to handle them at about the time initial set was taking place and it was felt that molds that were not rigid might allow stresses to be set up within the specimen.

The procedure followed in fabricating the specimens was to mix just sufficient mortar for one test specimen at a time. A well graded concrete sand was used in the mortar together with sufficient water to produce a plastic consistency. In each series, the water cement ratio was maintained constant. However, due to slight differences in grading of sand used in the different series, it was necessary to slightly vary the water cement ratio from series to series. The maximum difference did not exceed 0.02 by weight. The mortar was puddled into the molds with the gloved fingers, after which the surface was struck off with a single stroke of a steel blade. No troweling was done. Immediately after molding, the specimens were weighed, these weights being taken as the initial weights from which the moisture losses were computed.

In all cases where burlap was applied immediately after molding (except where it was sealed in as in method 1), the specimens were not placed immediately in the humidity controlled curing cabinet, but were placed in an oven maintained at 100°F. ± 2° F. but without humidity control. Where burlap was applied 1 or 3 hours after molding, the specimens were placed in the humidity controlled cabinet until the burlap was applied after which they were placed in the oven. This was necessary because the procedure in the case of burlap curing required that the material be kept saturated for the entire time of application. This would have made it impossible to maintain a constant humidity in the cabinet.

The burlap cover was kept saturated by immersing an overhanging end in a pan of water. It was found that to insure even and continuous saturation over the entire surface of the specimen it was necessary to use three layers of burlap. This procedure, which is of course not ordinarily followed in the field, was deemed necessary to insure a uniform moisture condition in all specimens during the curing period. It was necessary to keep the specimens wet in this manner to avoid the necessity of opening the oven doors continuously for the purpose of sprinkling.

At the conclusion of the burlap curing the specimens were removed from the oven, the burlap removed and the specimens immediately placed in the curing cabinet (where burlap curing only was involved) or covered with the final curing material and then placed in the cabinet.

In cases where burlap was not involved, the procedure was to place the specimen immediately after molding in the humidity controlled cabinet, removing it for the purpose of applying the curing material, after which it was replaced in the cabinet for the duration of the test.

At the conclusion of the exposure period the specimens were removed from the cabinet, the molds removed and the specimens immersed in water for 2 days prior to testing for flexural strength. For series A to D, inclusive, the age at test was therefore 9 days whereas for series E it was 30 days. To facilitate absorption of water, the upper and lower surfaces of each specimen were rubbed with a carborundum stone prior to immersion. This procedure was followed in an effort to place all specimens in a uniform condition, insofar as contained moisture was concerned, prior to test. As will be discussed in detail later, this apparently was not accomplished under all conditions, possibly accounting for certain discrepancies in the strength results observed.

Flexure tests were made at the conclusion of the 2-day resaturation period, the load being applied at the center of a 9-in. span, with the top surface as molded in tension.

The rate at which specimens cured by the different methods gave up water at various periods from time of molding up to and including 7 days' exposure, the flexural strengths of 9 days and the "relative efficiency," from the standpoint of both water retention and strength are shown in Tables 3 to 6, inclusive, except that Table 5 (series C) contains no data on relative efficiency because in this series no values were obtained on "no-cure" specimens (method No. 2). Corresponding values for 28-day exposure are shown in Table 7.

Relative efficiency as used in this report is a value which represents the comparative effectiveness of the particular method involved on the basis of 100 for the ideal cure (method 1a) and 0 for no-cure (method 2). Thus, in series A, Table 3, the strength of the perfectly cured specimens averaged 1,018 p.s.i., whereas, the no-cure specimens averaged 561 p.s.i. The difference, 457 p.s.i., may be considered as representing the gain in strength which was attained through ideal curing. On this basis, method 3a, 24-hr. burlap curing, with a strength of 861 p.s.i. had a relative efficiency of 66. Values for relative efficiency based on water loss were computed in the same manner. Thus, from Table 3 it will be noted that the specimens given no-curing (method 2) averaged 27 per cent moisture loss at 7 days, whereas, the specimens cured continuosuly under burlap (method 1a) gained 2 per cent in weight. By giving this value a rating of 100 and no-cure a rating of 0, the relative efficiency of, say, method 3a, with a moisture loss of 10 per cent, was found to be 59. This method of rating the efficiency of the various curing procedures is considered more satisfactory than expressing the result as a ratio of the value for ideal curing because it is a measure based on the *difference* in result between ideal curing and no-curing, whereas, the latter expresses the result in terms of ideal curing only.

The relative efficiencies of the several curing procedures are also shown in Tables 8 to 12, inclusive, to facilitate comparisons between similar methods as well as to provide ready comparison of the results obtained for the same procedure in different series.

DISCUSSION

In the following discussion of the results shown in Tables 3 to 7, inclusive, consideration will be given first to the rate at which specimens cured in various ways lost water during the exposure period. In general, the values obtained for the various methods checked very closely from series to series. The data also show that the different methods varied considerably in their ability to retain moisture. As would be expected, burlap covering applied immediately and kept saturated, 3a, 4a and 5a, not only retained all of the mixing water during the entire period of application, but also added water in amounts of from 1 to 4 per cent. However, as soon as the burlap was removed, the specimens started losing water, the amount retained at the end of 7 days depending upon the duration of the burlap curing.

It will also be observed that, in cases where the initial curing material was not applied until 3 hours after molding (methods in which the letter c follows the numeral), the specimens lost from 3 to 5 per cent of the mixing water before they were covered. Where burlap was, used and removed at the end of 24 hours (as, in methods 6c and 9c).

TABLE 3—SERIES A. RESULTS OF 7-DAY TESTS. ALL RESULTS AVERAGE OF FIVE TESTS

Method No.	Type of Curing	Procedure				Water Remaining in Specimens at Age Indicated*						Flexural Strength	Relative Efficiency		Method No.
		Burlap		Final Curing Material		Hours	Days						Based on		
		Applied after Molding	Duration of Application	Applied after Molding	Duration of Application	3	1	2	3	4	7		Water Loss	Strength	
		Hours	Days	Hours	Days	%	%	%	%	%	%	p.s.i.			
1a	Burlap	0	7	—	—	—	—	—	—	—	102	1,018	100	100	1a
2	None	—	—	—	—	97	79	77	76	74	73	561	0	0	2
3a	Burlap	0	1	—	—	—	101	95	93	92	90	861	59	66	3a
4a	Burlap	0	2	—	—	—	—	102	98	96	94	988	72	93	4a
5a	Burlap	0	3	—	—	—	—	—	102	99	97	996	83	95	6a
6a	Paper A with burlap	0	1	24	6	—	101	101	100	100	100	1,025	93	102	6a
7a	Sodium silicate with burlap	0	1	24	6	—	101	98	96	95	94	908	72	76	7a
8a	Calcium chloride with burlap	0	1	24	6	—	101	96	95	94	90	1,022	58	101	8a
9a	Liquid material A with burlap	0	1	24	6	—	101	101	100	100	100	1,055	93	108	9a
11a	Calcium chloride admixture with burlap	0	1	†	†	—	101	97	96	95	92	947	66	84	11a
12c	Curing blanket	—	—	3	3	96	—	—	90	89	86	740	45	39	12c
13c	Paper A	—	—	3	7	96	96	96	95	95	95	769	76	46	13c
14c	Paper B	—	—	3	7	97	96	95	94	94	94	768	72	45	14c
15c	Paper C	—	—	3	7	97	97	97	96	95	95	808	76	54	15c
16c	Paper D	—	—	3	7	96	84	82	80	80	77	571	14	2	16c
17c	Paper E	—	—	3	7	96	84	82	81	80	78	634	17	16	17c
18c	Paper F	—	—	3	7	95	84	82	80	79	77	610	14	11	18c
19c	Liquid material A	—	—	3	7	95	96	96	96	95	94	791	72	50	19c
20c	Liquid material B	—	—	3	7	96	95	95	93	93	92	762	66	44	20c
21c	Liquid material C	—	—	3	7	96	90	89	88	87	85	673	41	25	21c
22c	Liquid material D	—	—	3	7	96	94	93	93	92	92	724	66	36	22c

*Based on total water used.

†Mixing water contained 2 per cent calcium chloride.

TABLE 4—SERIES B. RESULTS OF 7-DAY TESTS. ALL RESULTS AVERAGE OF FIVE TESTS

Method No.	Type of Curing	Procedure: Burlap: Applied after Molding	Procedure: Burlap: Duration of Application	Procedure: Final Curing Material: Applied after Molding	Procedure: Final Curing Material: Duration of Application	Water Remaining in Specimens at Age Indicated*: Hours: 1	Hours: 3	Days: 1	Days: 2	Days: 3	Days: 4	Days: 7	Flexural Strength	Relative Efficiency: Based on: Water Loss	Relative Efficiency: Based on: Flexural Strength	Method No.
		Hours	Days	Hours	Days	%	%	%	%	%	%	%	p.s.i.			
1a	Burlap	0	7	—	—	—	—	—	—	—	—	103	924	100	100	1a
2	None	—	—	—	—	—	—	73	71	69	68	65	574	0	0	2
4a	Burlap	0	2	—	—	—	—	—	102	96	94	91	879	68	87	4a
4b	Burlap	1	2	—	—	99	—	—	99	92	90	87	841	58	76	4b
4c	Burlap	3	2	—	—	—	96	—	103	95	93	90	827	66	72	4c
4c-1†	Burlap sprinkled intermittently	3	2	—	—	—	96	—	99	95	93	90	798	66	66	4c-1
4c-2‡	Burlap sprinkled intermittently	3	2	—	—	—	96	—	90	86	85	82	714	45	37	4c-2
5a	Burlap	0	3	—	—	—	—	—	—	104	99	95	910	79	96	5a
5b	Burlap	1	3	—	—	99	—	—	—	99	94	90	907	66	95	5b
5c	Burlap	3	3	—	—	—	96	—	—	102	96	92	859	71	81	5c
5c-1†	Burlap sprinkled intermittently	3	3	—	—	—	97	—	—	99	94	91	885	68	92	5c-1
5c-2‡	Burlap sprinkled intermittently	3	3	—	—	—	96	—	—	90	89	85	698	53	32	5c-2
6a	Paper A with burlap	0	1	24	6	—	—	102	102	101	101	100	894	92	91	6a
6b	Paper A with burlap	1	1	24	6	99	—	99	99	99	98	97	956	84	109	6b
6c	Paper A with burlap	3	1	24	6	—	96	103	102	101	101	100	925	92	100	6c
13b	Paper A	—	—	1	7	99	—	97	96	96	95	94	855	76	80	13b
13c	Paper A	—	—	3	7	—	96	95	95.	94	94	93	838	74	75	13c
19b	Liquid material A	—	—	1	7	99	—	92	89	88	87	85	773	53	57	19b
19c	Liquid material A	—	—	3	7	—	96	95	95	94	94	93	849	74	79	19c
20b	Liquid material B	—	—	1	7	99	—	92	90	88	87	84	807	50	67	20b
20c	Liquid material B	—	—	3	7	—	95	94	94	93	92	92	919	71	99	20c
21b	Liquid material C	—	—	1	7	99	—	85	82	80	79	76	657	29	24	21b
21c	Liquid material C	—	—	3	7	—	95	88	86	84	83	81	733	42	45	21c

*Based on total water used. †Average of three tests. Burlap kept constantly in a moist condition.
‡Average of two tests. Burlap allowed to become practically dry before each sprinkling.

all of this water was regained during the first day after application, However, where no burlap was used (mothods 13c to 22c, inclusive) the loss was permanent, the surface sealing materials being unable to supply moisture lost during this period. In series B, Table 4, tests were run with burlap applied 1 hour after molding as well as after 3 hours (4b, 5b, and 6b). For some reason, as yet unexplained, this procedure did not result in adding water to the specimens, the amount of contained moisture being exactly the same at the end of the burlap curing period as when it was applied.

It will be noted that none of the surface sealing materials was completely effective in retaining all of the mixing water throughout the 7-day exposure period. For these materials moisture losses varied from 1 to 3 per cent for the most effective materials to as much as 25 per cent for the poorer materials. Furthermore, for a given material, this loss was about the same whether the material was used with or without burlap. When exposure in the curing cabinet was carried to 28 days (Table 7), further loss in moisture was observed in every case, the amounts ranging from 2 to 11 per cent, depending upon the material. In general, papers A, B, and C and liquid curing materials A and B were the most efficient of the surface seals in retaining water, with sodium silicate, the curing blanket and liquid curing materials C and D intermediate, and papers D, E, and F, the least efficient. However, as stated, none of the surface sealing methods studied was completely effective in retaining moisture during the period of application.

COMPARISON OF VARIOUS CURING MATERIALS ON BASIS OF RELATIVE EFFICIENCY

The relative efficiencies of the several curing procedures based on both water loss and flexural strength after 7 days' exposure, as given in Tables 3 to 6, inclusive, have been regrouped in Tables 8 to 12, inclusive, to compare more rapidly the effect of varying the details of applying methods of a similar nature as well as to facilitate comparisons of the results of each method from series to series.

In studying these data the reader is cautioned against drawing conclusions regarding the comparative values of the different methods based on comparisons of individual relative efficiencies. This applies particularly to efficiencies based on flexural strength results. As will be noted from the tables, these values for a given method varied considerably from series to series. The variations were more pronounced in the case of the surface sealing materials, such as paper and the liquid curing materials than where methods involving burlap only were used. Furthermore, they seem to follow a general trend in that

TABLE 5—SERIES C. RESULTS OF 7-DAY TESTS. ALL RESULTS AVERAGE OF SIX TESTS

Method No.	Type of Curing	Procedure: Burlap: Applied after Molding	Procedure: Burlap: Duration of Application	Procedure: Final Curing Material: Applied after Molding	Procedure: Final Curing Material: Duration of Application	Water Remaining in Specimens at Age Indicated*: Hours: 3	Water Remaining in Specimens at Age Indicated*: Days: 1	2	3	4	7	Flexural Strength	Method No.
		Hours	Days	Hours	Days	%	%	%	%	%	%	p.s.i.	
1a	Burlap	0	7	—	—	—	—	—	—	—	104	843	1a
6c	Paper A with burlap	3	1	24	6	96	102	102	102	102	100	810	6c
9c	Liquid material A with burlap	3	1	24	6	96	104	103	102	102	100	823	9c
10c	Liquid material B with burlap	3	1	24	6	95	102	101	101	100	98	826	10c
13c	Paper A	—	—	3	7	95	94	94	94	93	92	786	13c
19c	Liquid material A	—	—	3	7	95	93	95	94	94	94	818	19c
20c	Liquid material B	—	—	3	7	95	94	93	92	92	90	786	20c

*Based on total water used.

TABLE 6—SERIES D. RESULTS OF 7-DAY TESTS. ALL RESULTS AVERAGE OF FIVE TESTS

Method No.	Type of Curing	Procedure: Burlap: Applied after Molding	Procedure: Burlap: Duration of Application	Procedure: Final Curing Material: Applied after Molding	Procedure: Final Curing Material: Duration of Application	Water Remaining in Specimens at Age Indicated*: Hours: 3	Water Remaining in Specimens at Age Indicated*: Days: 1	2	3	4	7	Flexural Strength	Relative Efficiency: Based on: Water Loss	Relative Efficiency: Based on: Strength	Method No.
		Hours	Days	Hours	Days	%	%	%	%	%	%	p.s.i.			
1a	Burlap	0	7	—	—	—	—	—	—	—	104	897	100	100	1a
2	None	—	—	—	—	96	76	73	71	71	67	597	0	0	2
3a	Burlap	0	1	—	—	—	103	94	91	90	86	808	51	70	3a
4a	Burlap	0	2	—	—	—	—	104	98	96	92	875	68	93	4a
5a	Burlap	0	3	—	—	—	—	—	104	99	95	909	76	104	5a
6a	Paper A with burlap	0	1	24	6	—	103	102	102	102	100	943	89	115	6a
7a	Sodium silicate with burlap	0	1	24	6	—	103	96	94	92	89	865	59	89	7a
8a	Calcium chloride with burlap	0	1	24	6	—	100	95	93	91	88	867	57	90	9a
9a	Liquid material A with burlap	0	1	24	6	—	102	101	101	101	99	943	86	115	9a
11a	Calcium chloride admixture with burlap	0	1	†	†	—	103	95	93	91	88	845	57	83	11a
12c	Curing blanket	—	—	3	3	96	—	—	89	87	84	741	46	48	12c
13c	Paper A	—	—	3	7	96	95	95	94	94	93	785	70	63	13c
14c	Paper B	—	—	3	7	95	95	94	94	94	93	780	70	61	14c
15c	Paper C	—	—	3	7	96	95	95	95	95	94	805	73	69	15c
16c	Paper D	—	—	3	7	96	81	78	76	75	71	623	11	9	16c
17c	Paper E	—	—	3	7	95	83	80	78	77	74	621	19	8	17c
18c	Paper F	—	—	3	7	95	83	79	78	76	73	600	16	1	18c
19c	Liquid material A	—	—	3	7	96	96	95	95	95	94	822	73	75	19c
20c	Liquid material B	—	—	3	7	96	94	93	92	92	90	759	62	54	20c
21c	Liquid material C	—	—	3	7	96	92	89	88	87	84	678	46	27	21c
22c	Liquid material D	—	—	3	7	95	93	91	91	90	89	719	59	41	22c

*Based on total water used.

†Mixing water contained 2 per cent calcium chloride.

*Based on total water used. †Mixing water contained 2 per cent calcium chloride.

TABLE 7—SERIES E. RESULTS OF 28-DAY TESTS. RESULTS AVERAGE OF FIVE TESTS EXCEPT AS NOTED

Method No.	Type of Curing	Procedure: Burlap, Applied after Molding	Procedure: Burlap, Duration of Application	Procedure: Final Curing Material, Applied after Molding	Procedure: Final Curing Material, Duration of Application	Water Remaining in Specimens at Age Indicated*: Hours 3	Days 1	Days 2	Days 3	Days 4	Days 7	Days 14	Days 21	Days 28	Flexural Strength	Relative Efficiency: Based on Water Loss	Relative Efficiency: Based on Strength	Method No.
		Hours	Days	Hours	Days	%	%	%	%	%	%	%	%	%	p.s.i.			
1a	Burlap	0	28	—	—	—	—	—	—	—	—	—	—	102	1,156	100	100	1a
2	None	—	—	—	—	95	75	72	72	71	68	64	62	59	558	0	0	2
‡ 3a	Burlap	0	1	—	—	—	101	94	92	91	90	84	83	81	756	51	42	3a
** 3c	Burlap	3	1	—	—	96	103	93	92	92	87	84	82	79	773	47	31	3c
‡ 4a	Burlap	0	2	—	—	—	—	103	98	97	95	91	89	86	834	63	56	4a
** 4c	Burlap	3	2	—	—	97	—	102	97	96	92	89	87	83	822	56	38	4c
‡ 5a	Burlap	0	3	—	—	—	—	—	102	99	97	91	90	88	810	67	52	5a
** 5c	Burlap	3	3	—	—	96	—	—	103	98	95	91	88	86	841	63	42	5c
‡ 6a	Paper A with burlap	0	1	24	27	—	102	101	101	100	100	99	97	96	1,216	86	125	6a
** 6c	Paper A with burlap	3	1	24	27	96	103	102	102	102	100	99	98	96	1,185	86	96	6c
‡ 7a	Sodium silicate with burlap	0	1	24	27	—	101	97	96	94	93	88	87	85	773	60	46	7a
** 7c	Sodium silicate with burlap	3	1	24	27	96	104	98	97	96	93	90	88	85	819	60	38	7c
‡ 8a	Calcium chloride with burlap	0	1	24	27	—	99	97	95	95	93	89	87	85	909	60	69	8a
** 8c	Calcium chloride with burlap	3	1	24	27	96	103	98	97	95	92	89	85	81	908	51	52	8c
‡ 9a	Liquid material A with burlap	0	1	24	27	—	101	100	100	99	99	96	96	94	1,106	81	106	9a
** 9c	Liquid material A with burlap	3	1	24	27	96	101	100	98	98	96	94	93	91	953	74	60	9c
‡11a	Calcium chloride admixture with burlap	0	1	†	†	—	104	98	96	95	93	89	87	84	770	58	46	11a
**11c	Calcium chloride admixture with burlap	3	1	†	†	96	104	96	95	94	90	87	85	83	848	56	44	11c
12c	Curing blanket	—	—	3	3	96	—	—	89	88	85	81	79	76	682	40	21	12c
13c	Paper A	—	—	3	28	97	96	95	94	94	93	92	91	90	799	72	40	13c
14c	Paper B	—	—	3	28	96	95	93	93	93	92	90	88	86	780	63	37	14c
15c	Paper C	—	—	3	28	96	95	95	95	95	94	93	93	92	866	77	52	15c
16c	Paper D	—	—	3	28	95	82	78	78	76	74	70	67	65	589	14	6	16c
17c	Paper E	—	—	3	28	95	83	80	79	78	75	72	69	66	563	16	2	17c
18c	Paper F	—	—	3	28	95	81	78	77	76	74	70	67	65	564	14	2	18c
19c	Liquid material A	—	—	3	28	96	94	94	94	93	92	91	90	89	891	70	56	19c
20c	Liquid material B	—	—	3	28	95	93	90	91	90	88	85	83	80	692	49	23	20c
21c	Liquid material C	—	—	3	28	95	88	84	83	82	79	75	73	71	610	28	10	21c
22c	Liquid material D	—	—	3	28	95	89	88	88	88	86	85	85	82	726	53	29	22c

*Based on total water used. †Mixing water contained 2 per cent calcium chloride. ‡Average of two rounds of tests. **Average of three rounds of tests.
***The relative efficiency based on strength was computed for each round separately from the strength of the "standard" and "no cure" specimens in that round and the average of these values is the relative efficiency shown.

the efficiencies calculated from the results of tests made in series A are, in general, low, those obtained from series B, in general, high, while the results obtained from series D are as a rule intermediate. As previously mentioned, these discrepancies may possibly be the result of variations in the moisture condition of the specimens at time of test.

As is well known, the distribution of moisture within a flexure specimen at time of test will appreciably affect its strength. In general, if the shell of the specimen contains more moisture than the core (a condition usually resulting from incomplete saturation after drying), the extreme fibers will be in compression and the observed breaking load will be higher than the true value. On the other hand, if the shell contains less moisture than the core (a condition usually associated with incomplete drying out) the observed value will be lower than the true value. Due to the fact that these specimens were tested after an immersion period during which they may not have taken up sufficient water to secure complete saturation, it is possible that the comparatively high relative strengths obtained in certain series may have resulted from incomplete saturation of the specimens.

In preparing the specimens for test, every effort was made to insure complete distribution of moisture. This, of course, is the only condition under which flexure tests of concrete should be made. However, inspection of the fractured specimens indicated that in many instances complete saturation was not accomplished even after 48 hours immersion. The ideally cured specimens (method 1a) were, of course, thoroughly saturated when tested. The no-cure specimens, as well as those cured with the least efficient surface sealing materials, due to their lack of density, also took up water more readily upon resaturation than the specimens cured by the more efficient surface sealing materials. Therefore, if the low ratings for the various curing materials were obtained because of more complete saturation, these ratings may possibly be considered to represent more nearly the true curing effect than where high ratings for the same method are shown.

In spite of wide variations in strength results in the different series, it is felt that the strength data are significant in that they indicate definite trends insofar as the general effectiveness of the several classes of curing materials are concerned. These trends will be pointed out in the discussion of Tables 8 to 12, inclusive, which follows:

In Table 8 the results of varying the time of application after molding and the duration of curing with wet burlap are given. It

TABLE 8—EFFECT OF TIME OF APPLICATION AND DURATION OF CURING USING WET BURLAP

Method No.	Procedure	Relative Efficiency							
		Based on Water Loss				Based on Strength			
		Series				Series			
		A	B	D	Ave.	A	B	D	Ave.
3a	Burlap applied immediately after molding for 1 day	59	—	51	55	66	—	70	68
4a	Burlap applied immediately after molding for 2 days	72	68	68	69	93	87	93	91
5a	Burlap applied immediately after molding for 3 days	83	79	76	79	95	96	104	98
4b	Burlap applied 1 hour after molding for 2 days	—	58	—	58	—	76	—	76
5b	Burlap applied 1 hour after molding for 3 days	—	66	—	66	—	95	—	95
4c	Burlap applied 3 hours after molding for 2 days	—	66	—	66	—	72	—	72
5c	Burlap applied 3 hours after molding for 3 days	—	71	—	71	—	81	—	81
4c-1	Burlap applied 3 hours after molding for 2 days; sprinkled intermittently; kept continuously wet	—	66	—	66	—	66	—	66
5c-1	Burlap applied 3 hours after molding for 3 days; sprinkled intermittently; kept continuously wet	—	68	—	68	—	92	—	92
4c-2	Burlap applied 3 hours after molding for 2 days; sprinkled intermittently; allowed to become dry between wettings	—	45	—	45	—	37	—	37
5c-2	Burlap applied 3 hours after molding for 3 days; sprinkled intermittently; allowed to become dry between wettings	—	53	—	53	—	32	—	32

TABLE 9—COMPARISON OF VARIOUS CURING PAPERS AND EFFECT OF TIME OF APPLICATION

Method No.	Type of Paper	Procedure	Relative Efficiency							
			Based on Water Loss				Based on Strength			
			Series				Series			
			A	B	D	Ave.	A	B	D	Ave.
13b	A	Paper applied 1 hour after molding for 7 days	—	76	—	76	—	80	—	80
13c	A	Paper applied 3 hours after molding for 7 days	76	74	70	73	46	75	63	61
14c	B	Paper applied 3 hours after molding for 7 days	72	—	70	71	45	—	61	53
15c	C	Paper applied 3 hours after molding for 7 days	76	—	73	74	54	—	69	62
16c	D	Paper applied 3 hours after molding for 7 days	14	—	11	12	2	—	9	6
17c	E	Paper applied 3 hours after molding for 7 days	17	—	19	18	16	—	8	12
18c	F	Paper applied 3 hours after molding for 7 days	14	—	16	15	11	—	1	6

will be noted that, regardless of the time elapsing before the application of the burlap, the efficiency of this method of curing is increased as the duration of the application is increased. This is true for both water loss and strength. For instance, method 3a, where the burlap was applied immediately and remained in place for 1 day, had a relative efficiency based on water loss of 55 and on strength of 68. When the same material was allowed to remain in place 3 days (method 5a) the efficiency based on water loss was raised to 79 and that based on strength to 98. The effect of delaying the application of the burlap

TABLE 10—COMPARISON OF VARIOUS LIQUID CURING MATERIALS AND EFFECT OF TIME OF APPLICATION

Method No.	Type Liquid Curing Material	Procedure	Relative Efficiency							
			Based on Water Loss				Based on Strength			
			Series				Series			
			A	B	D	Ave.	A	B	D	Ave.
19b	A	Liquid applied 1 hour after molding for 7 days	—	53	—	53	—	57	—	57
20b	B	Liquid applied 1 hour after molding for 7 days	—	50	—	50	—	67	—	67
21b	C	Liquid applied 1 hour after molding for 7 days	—	29	—	29	—	24	—	24
19c	A	Liquid applied 3 hours after molding for 7 days	72	74	73	73	50	79	75	68
20c	B	Liquid applied 3 hours after molding for 7 days	66	71	62	66	44	99	54	66
21c	C	Liquid applied 3 hours after molding for 7 days	41	42	46	43	25	45	27	32
22c	D	Liquid applied 3 hours after molding for 7 days	66	—	59	62	36	—	41	38

was to lower the efficiency as measured by strength (compare methods 4a, 4b, and 4c, for 2-day curing and 5a, 5b, and 5c, for 3-day curing). The same trends appear when the efficiency is measured by water loss except that in the case of both 2-day and 3-day curing, the amount of water remaining at the end of the 7-day period was somewhat less when the burlap was applied at 1 hour after molding than when applied after 3 hours. This reversal of trend has already been commented upon.

In methods 4c-1, 5c-1, 4c-2, and 5c-2, the effect of continuous and intermittent sprinkling is shown. Comparing 4c with 4c-1 and 5c with 5c-1, it will be noted that about the same results were obtained when the curlap was kept wet by sprinkling as when continuously saturated by keeping an end of the covering immersed in water. The effectiveness based on both water loss and strength was, however, seriously affected when the burlap was allowed to dry out between the sprinklings. (Compare 4c with 4c-2 and 5c with 5c-2). These data illustrate the importance of maintaining a continuous wet covering when burlap is used.

The results obtained with the six curing papers are shown in Table 9. Papers A, B, and C, seem to be about equally effective as is also true for papers D, E, and F, except that the latter three papers show much poorer results. Papers D, E, and F, in fact, gave strengths little better than no curing at all. The effect of time of application for paper A may be noted by comparing methods 13b and 13c. It will be observed that the efficiency of the paper, especially from the standpoint of strength, is somewhat less when the time of application is delayed.

TABLE 11—COMPARISON OF PAPER AND LIQUID CURING MATERIALS WITH AND WITHOUT PRELIMINARY BURLAP, AND EFFECT OF TIME OF APPLICATION

Method No.	Procedure	Relative Efficiency							
		Based on Water Loss				Based on Strength			
		Series				Series			
		A	B	D	Ave.	A	B	D	Ave.
6a	Burlap applied immediately followed in 24 hours by paper A for 6 days	93	92	89	91	102	91	115	103
6b	Burlap applied 1 hour after molding, followed in 24 hours by paper A for 6 days	—	84	—	84	—	109	—	109
6c	Burlap applied 3 hours after molding, followed in 24 hours by paper A for 6 days	—	92	—	92	—	100	—	100
13b	Paper A applied 1 hour after molding for 7 days	—	76	—	76	—	80	—	90
13c	Paper A applied 3 hours after molding for 7 days	76	74	70	73	46	75	63	61
9a	Burlap applied immediately followed in 24 hours by liquid material A	93	—	86	90	108	—	115	112
19b	Liquid material A applied 1 hour after molding	—	53	—	53	—	57	—	57
19c	Liquid material A applied 3 hours after molding	72	74	73	73	50	79	75	68

TABLE 12—MISCELLANEOUS CURING MATERIALS

Method No.	Type of Curing	Procedure	Relative Efficiency					
			Based on Water Loss			Based on Strength		
			Series			Series		
			A	D	Ave.	A	D	Ave.
7a	Sodium silicate with burlap	Burlap applied immediately followed in 24 hours by sodium silicate	72	59	66	76	89	82
8a	Calcium chloride with burlap	Burlap applied immediately followed in 24 hours by a surface application of calcium chloride	59	57	58	101	90	96
11a	Calcium chloride admixture with burlap	Burlap applied immediately for 1 day. Two per cent calcium chloride admixture added to mixing water	66	57	62	84	83	84
12c	Curing blanket	Curing blanket applied 3 hours after molding for 3 days	45	46	46	39	48	44

Comparisons of the effectiveness of the various liquid curing compounds when used without burlap; that is, as recommended by the manufacturers, may be made from Table 10. It will be observed that liquid materials A and B were considerably more effective than materials C and D. However, in no case except one does the average efficiency approach that obtained by, say, the 3-day burlap curing shown in Table 8, method 5a. The exception is method 20c, series B. This is an illustration of an unusually high value which may have resulted from incomplete saturation of the specimens. It will be observed also that in every case, except one, the relative efficiency of the liquid curing materials is increased by delaying the application to 3 hours after molding. This is just the reverse of the trend shown for curing with paper A (table 9). This increased efficiency may possibly be accounted for by the fact that when the liquid material was sprayed on at the end of 3 hours, surface moisture had disappeared to an extent

which permitted a more perfect seal than when the material was applied at the end of 1 hour. The results emphasize the necessity of watching this detail carefully when applying such materials in the field.

Table 11 permits a comparison of the results obtained with paper A and liquid curing material A when used with and without an initial curing of wet burlap. It will be observed from the data that in both cases the efficiency of the surface sealing material is materially increased by the use of burlap. Additional data along this line are shown in Table 5 (series C). The results of these tests were not included in Table 11 because, due to the omission of the no-cure method, no calculations of relative efficiency could be made. The results indicate that when the application of the burlap is delayed for 3 hours, the strengths of the specimens cured without burlap (13c, 19c, and 20c) are very nearly as high as when burlap was used. However, due to the fact that the saturated burlap returned to the specimen water lost during the first 3 hours, the total water retained at the end of 7 days was somewhat higher when burlap was used than when the paper and liquid curing materials were used as recommended by the manufacturers. In general, the conclusion is that for best results such surface sealing materials as paper, liquid asphalt, etc., should be used following a 24-hour application of wet burlap.

In Table 12 are shown the results of tests with sodium silicate, calcium chloride and the special curing blanket.

In testing these materials the general practice as used in the field was followed. For sodium silicate the results indicate an effectiveness somewhat less, in general, than 3-day burlap and considerably less than the best waterproof paper or liquid curing materials used with burlap.

The results with calcium chloride are rather conflicting. For instance, the strengths obtained in the surface application method are somewhat higher than would be expected from the water losses indicated. It is apparent that, at the low relative humidity to which these specimens were subjected (32 per cent) the calcium chloride withdrew water from the specimen rather than from the air. The strengths, however, are high. The admixture did not seem to provide any better water retaining properties than many of the surface seals. Moreover, under these conditions, the strengths of the specimens containing the admixture were low. This also may have been due to the low humidity and high temperature (100° F.) to which the specimens were exposed.

The special curing blanket, which was wet once when applied and remained in place 3 days, was low in efficiency as measured by both strength and water loss. Attention is directed to the fact that this blanket was of burlap and jute and it should not be confused with the cotton mats which, as previously stated, have proven highly effective for curing purposes. Neither should the results obtained with the jute blanket be regarded as representative of what would have been obtained had the blanket been wet at sufficiently frequent intervals to have kept it in a continuously moist condition.

Relative efficiencies of the various curing materials based on water retention and strength at the end of 28 days, are shown in Table 7. Attention is called to the fact that, for methods 3 to 11, inclusive (methods involving the use of burlap), the results are the average of only two rounds in the case of (a) series, and three rounds in the case of (c) series, instead of five rounds as in all other cases.

With this limitation in mind, it may be noted that all of the methods involving burlap only, that is 3a to 5c, inclusive, have low ratings as compared to the corresponding results at 7 days. Furthermore, the beneficial effects of 3-day burlap curing as compared to 1-day curing appear to be somewhat less pronounced. Waterproof paper A following burlap (6a and 6c) developed high strength at 28 days. Attention is called to the fact that the paper remained in place for the full 28-day period. The same material without burlap (13c) showed a relative efficiency of only 40 as regards strength. Liquid curing material A gave high strengths when the burlap was applied immediately (9a) but showed a comparatively low relative efficiency when the application of the burlap was delayed 3 hours (9c). Without burlap, liquid material A (19c) showed a rating of 56, only slightly lower than the combination in which the burlap was applied after 3 hours. These trends seem to parallel in general the indications at 7 days (see Table 5, series C and Table 11). With respect to burlap curing as compared with the seals, it might be pointed out that in this high temperature, low humidity atmosphere, the curing with burlap was discontinued at 3 days, whereas, it continued to some extent under the seals that were effective.

The relative efficiency of the methods employing sodium silicate, calcium chloride and the curing blanket (methods 7, 8, 11, and 12) are about the same at 28 days as at 7 days, when judged from the standpoint of water retention, but are much lower when considered from the standpoint of strength. However, the small number of

specimens represented make any comparisons involving these methods of doubtful value.

The most significant point in connection with the 28-day test data lies in the fact that in only two cases do the strength ratings anywhere near approach that of method 1a. These methods, burlap applied immediately followed by waterproof paper A (6a), and burlap applied immediately followed by liquid bituminous material A (9a), provide the most nearly perfect continuous seals of any of the methods tested, thus emphasizing the conclusion that the greatest curing efficiency is provided by those procedures which protect the concrete against moisture loss to the greatest extent.

CONCLUSIONS

The results secured in this investigation seem to warrant the following general conclusions:

(*a*) *As regards burlap used alone:*

(1) The effectiveness of burlap curing is increased by lengthening the duration of application.

(2) The effectiveness of burlap curing is decreased by increasing the time elapsing between the placing of the concrete and the application of the burlap.

(3) Burlap curing is not as effective when sprinkled intermittently as when kept continuously saturated.

(*b*) *As regards surface sealing materials:*

(1) The effectiveness of such materials as waterproof paper and liquid curing materials applied with a spray is materially increased when preceded by a 24-hour application of wet burlap.

(2) The effectiveness of such membrane coverings as liquid curing materials A and B is materially improved by applying the covering 3 hours after molding as compared to an application made 1 hour after molding.

Discussion, to close in the September Supplement, should reach the A. C. I. Secretary by July 20.

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Discussion of a paper by Jackson and Kellermann:

Tests of Concrete Curing Materials*

BY DUFF A. ABRAMS, H. S. MATTIMORE, IRA PAUL, R. B. GAGE, HAROLD P. HAYDEN (AT THE CONVENTION) CARLL W. HUNT AND AUTHORS

CONVENTION DISCUSSION

Duff A. Abrams—Curing of concrete may be defined as the provision of an environment which is suitable for the setting and hardening of the mass. A satisfactory environment includes: (1) Ample moisture; (2) Proper temperature, (3) Freedom from premature stresses, (4) Time factor.

It has been long recognized that moisture is necessary for the continuing hydration of portland cement. For the past 30 years standard curing of cement or concrete specimens has consisted of exposure at about 70° F. in a moist room, in a fog-room or immersion in water. When drying is desired under controlled conditions, many experimentors have used air at 70° F. and a relative humidity of 50 per cent.

The method used by the authors discards both of these practices and adopts another combination of conditions which has never been used before so far as I am aware.

The type of cement used, the quantities of cement, sand and water which made up the 1-2 mortar mixes are not given, therefore we must assume that the cement, sand and water-cement ratio were the same for all tests.

It appears to be the authors' aim to offer the procedure given in the paper as the basis of a standard method of evaluating the effectiveness of curing methods, particularly as applied to concrete pavements.

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†Consulting Engineer, New York. This duscussion presented in part only at Am. Concrete Inst., convention New York, March 1, 1939.

The so-called "perfect" curing method consisted of covering the top of fresh 1-2 mortar 2 in. thick, which had been molded in a tight pan, with 2 layers of saturated burlap and sealing the entire assemblage inside a second metal pan and exposing at 100° F. for 7 days. Companion specimens are exposed to 100° F. and relative humidity of 32 per cent.

This must be construed as an accelerated test since these are not usual conditions for concrete road construction, and would be encountered only in the semi-arid states of the Southwest. The difficulties which bedevil an accelerated test are well illustrated by the autoclave test for constancy of volume of portland cements. This test has been under investigation for about 30 years and has been the subject of intensive study in this country during the past 5 years, yet there is now no agreement as to the interpretation of the values obtained.

All evidence available indicates that a 1-2 sand mortar mix will lose water much faster than a concrete with aggregates graded to say 2½ in.

Tests carried out by the writer on losses from free water surfaces and saturated aggregates of different sizes showed that on the basis of 100 per cent loss from free water in a unit of time, the saturated sand (0-4) showed 85 per cent loss, while the saturated sand and gravel mixture (0-1½) showed only 50 per cent loss. In other words the sand lost water about 1.7 times as fast as the combined aggregates. At 75° F. and humidity 60 per cent (which simulates field conditions), the relative values for water losses were about the same,

While the foregoing values are for aggregate only, a similar retarding effect would undoubtedly be reflected in the water loss of concrete containing coarse aggregates as compared with a sand mortar.

The authors have devised a method of computing efficiences which can best be characterized as fantastic. Out of the infinite combinations possible they have arbitrarily chosen one temperature (100° F.), two humidities (100 and 32 per cent) and one age (7 days). There is nothing in the report which gives any clew as to the efficiencies which would be secured for conditions which generally prevail during concrete road construction, say 75° F. and humidity 50 to 60 per cent.

The entire scheme of the authors' test method appears to be based on the dependence of strength on the quantity of mixing water retained in the mortar. While it is universally conceded that mixed concrete contains sufficient water for the hydration of the cement, it is not generally recognized that the distribution of this water is far from uniform throughout the depth of a pavement slab whose top is exposed to drying.

The authors begin with one problem (quantity of water retained by the various curing agents) and wind up with two conflicting answers (efficiencies for water loss and strength).

Logically the authors' method would require a zero strength to correspond with a zero water content. I know of no combination of temperature and humidity which will furnish a zero point on such a scale. The authors consider 65 to 73 per cent of the mixing water the zero point on their moisture scale for the 7-day tests and 59 per cent for the 28-day tests.

The zero of strength efficiencies is that given by the modulus of rupture of the specimens without any surface protection. However it will be noted that these zero efficiencies give modulus of rupture values from 561 to 597 p.s.i. at 9 days. Only a few years ago these would have been considered satisfactory values for the 28-day strength of road concrete.

We might expect that the "perfect" cure would give a perfect strength; however this is far from the case; the tables give numerous instances of strengths which are higher than the perfect-cure strength by as much as 25 per cent.

We commented above on having two answers to one problem; the following values bring out some inconsistencies of the tests:

Table No.	Method No.	Efficiency—Per Cent		
		Water Loss	Strength	Difference
3	9a	93	108	15
4	6b	84	109	25
6	5a	76	104	28
6	6a	89	115	27
6	9a	86	115	29
7	6a	86	125	39
7	9a	81	106	25

Here we find a spread of as high as 39 per cent between the two answers to the same problem.

In Table 3 the zero strength is 55 per cent of the strength of the perfect cure, while in Table 6 (duplicate tests made several months later), the zero strength was 67 per cent of the perfect cure.

In Table 3 the difference between zero strength and perfect strength is 457 p.s.i.; in duplicate tests in Table 6 it is 300 p.s.i.; a spread of over 50 per cent.

It is difficult to comment further on the strength tests without more details of the test methods; it seems likely that the sudden heating and cooling of the thin specimens may have caused incipient cracking of the exposed surface. The fact that the highest strengths were

always found for curing methods which would tend to minimize this effect, lends color to this view.

The perfect curing method, with its covering of wet burlap and cushion of air beneath the sealing pan would be least affected by this sudden change in temperature.

The rapid drying of the surface of the concrete during the first three hours while the mortar is still unset and before the paper or liquid curing materials were applied, would be expected to produce minute fissures in the surface which is later exposed to tension in the strength test. This no doubt accounts for the low values found for the modulus of rupture of these specimens.

The lack of co-ordination between the quantity of water retained and the modulus of rupture of the specimens (shown by plotting the values as abscissae and ordinates) is sufficient proof that the strength is influenced by factors other than retained water. The general trend of the plotted points shows a gain in modulus of rupture for each 1 per cent of mixing water retained after 7 days curing as follows:

Series A	18 p.s.i.
Series B	9 p.s.i.

Here we find 100 per cent variation in the indications of duplicate runs of the same series.

To consider only the case of liquid materials A and B (Tables 3 and 4), it seems that sealing materials which after application permit the loss of only 1 to 4 per cent of the mixing water at the end of 7 days, under the severe conditions of these tests, are doing about all that could be expected and all that practical conditions demand; yet these materials are rated at 74 to 66 per cent efficient on the basis of water loss. In these tests the authors have made the extraordinary mistake of *penalizing the curing materials for water losses which occurred before* the materials were applied.

A question may be raised as to whether a 28-day curing test under the standardized conditions mentioned at the beginning of this discussion would not have been preferable to the 7-day tests under abnormal temperature and humidity range used by the authors.

It appears that too much has been sacrificed for the convenience of the laboratory forces; small mortar specimens are convenient to mold, store and weigh, but what do they demonstrate with reference to concrete? Tests on concrete specimens approximating the mixes and dimensions of field practice would have been more convincing.

A careful study of the paper has convinced me that

(1) Curing tests on a 1-2 sand mortar slab 2 in. thich under the temperature and humidity conditions used are of doubtful value when applied to concrete pavements.

(2) The test methods do violence to each and every one of the factors which should be considered as desirable for a standard method of evaluating curing materials.

(3) There is much to be done before a method of this type can possibly be considered satisfactory for a standard test procedure.

*H. S. Mattimore**—We have very little, or no authentic data on efficiency of different types of curing. This paper of Mr. Jackson's is a contribution helpful to the engineer in furnishing knowledge on an essential procedure. It seems to me that a paper of this type should be received as such rather than picking small flaws that may exist, but do not affect the value of the research.

As to the 24-hr. burlap covering used prior to sealing, it seems to me it is an excellent precaution against moisture loss at the early periods. Also the early water curing will retard the rise in temperature which usually accompanies bituminous coverings applied at early stages.

The difference between the one- and three-hour liquid curing may be accounted for—in the three-hour period more liquid bituminous material can be applied to the pavement to form a more perfect seal than at the early ages. This is due to the fact that moisture exists on the surface during these early periods, and the amount of bituminous material that can be applied is thereby limited.

Ira Paul†—In New York State the Highway Department has confined its curing methods on concrete pavements essentially to two types of materials, one—the quilted covers, and the other to bituminous materials, either in the form of an asphalt emulsion or cut back.

Our specification for quilted covers states that the approved quilted covers shall consist of outer covering materials of burlap, cotton or other approved fabric, the bottom and top cover of which shall each weigh at least 6 ounces per square yard of fabric, and an inner layer of cotton batting or other approved material weighing at least 8 ounces per sq. yd. These mats shall be quilted to hold the inner cotton batting lining securely in place and shall be two feet greater in width, after shrinking, than the pavement slabs being cured.

The adjoining quilts shall be lapped at least 12 inches. The quilted covers are placed in a thoroughly wet condition directly on the pave-

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ment as soon as it is completed, and kept wet for at least 5 days on straight portland cement concrete laid from June 1 to Sept. 15, and for at least 7 days on that laid before June 1 and after Sept. 15. With high early strength cements the curing periods are 2 days, and 5 days respectively during the construction periods mentioned.

Compressive strength tests made on straight portland cement concrete cores taken from the pavement showed that when the curing period was cut down from 5 to 3 days the strength ratio was reduced to about 88 per cent.

As to bituminous curing material applied immediately after the brooming of the concrete surface, the compressive strength tests on such pavement cores were equal to that of the 5 day wet quilted mat curing. While we have no direct comparative strength tests of field cores on the same project, between the immediate bituminous curing method and that applied after 24 hours of wet burlap curing, no substantial differences in the compressive strength of concrete from many pavements cured by either of these methods has been noted.

In my field inspection of concrete roads, I have observed that there is a greater tendency for the slabs to check and crack where liquid bituminous curing materials are applied immediately after finishing. This is particularly noticeable on those pavements constructed during the hot summer months, when the air temperatures in the sun are 100° F. and over. This condition does not manifest itself on concrete roads constructed during the spring and fall months ofthe year with lower temperatures. The concrete pavement is substantially free of cracks, regardless of the time of year it is constructed, where either the wet quilted mat method is used or the liquid bituminous material is sprayed after a 24-hour application of wet burlap.

*R. B. Gage**—The writer has tried to digest the conclusions arrived at by Mr. Jackson in his discussion regarding the curing of concrete; but the relative value of the concrete having been judged by its tensile instead of by its compression strength, practically eliminates all the data collected by the New Jersey State Highway Department during the last 15 or 20 years. The behavior of the pavements and the service they have given, have been correlated and judged in conjunction with core records from the different types and grades of pavements.

Pavements do crack occasionally but they are much more prone to peel, scale and disintegrate, which can hardly be considered a failure in the tensile strength of the concrete.

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There appear to be several factors in the durability of pavement concrete not satisfactorily explained by laboratory tests. The New Jersey State Highway Department has known of these factors a number of years and efforts have been made to eliminate some of them as rapidly as possible. This is not always easy to accomplish, due to the fact that when a pavement exceeds its anticipated life, the contractors and the material producers suffer—out of work and short of orders.

Again, there appears to be a definite relation between the density of the concrete, its durability, and the unit volume used during preparation. The kneading and mixing action, appears to produce a denser and stronger concrete as the volume unit increases. The same is also true of the mixing time up to a certain point—regulated by the volume of the concrete. These factors apparently have been completely ignored by Mr. Jackson, since they cannot be properly regulated or controlled in the small batches used in this experiment.

The New Jersey State Highway Department has also noted for a number of years that our best, densest and strongest concrete is constructed either early in the spring or late in the fall, when the weather conditions are such that the initial set in the cement is greatly retarded. Where the temperature of this concrete has been kept between 32° to 40°, apparently the best results in both strength and durability have been secured.

This is apparently contrary to many of the specifications now in effect. To maintain such conditions during the hotter parts of the year, the bituminous method of curing should not be used, but a covering material that will retard the setting of the cement as much as possible. Again, the fact should not be overlooked that with perfect curing the quantity of water needed to combine with the cement to secure perfect hydration is small, and an excess not needed. This fact we have repeatedly proven by cores taken from pavements that were cured only a few hours. Yet the concrete had the desired strength and durability. Unless the subgrade is practically devoid of moisture, that necessary to cure the concrete will be absorbed from the subgrade. Even under adverse subgrade conditions, a surface covering mat will supply the moisture necessary to cure the concrete properly and prevent any surface checking or cracking.

Again, just why a mortar having the composition of 1 part of cement to 2 parts of sand by volume was used in this experiment is not explained; many specifications for highway pavements do not permit the use of so lean a mortar. Many of the failures that have taken

place in concrete pavements can be attributed to the use of too lean a mortar, too much mixing water, or a combination of these two factors. Fortunately, the State of New Jersey has not constructed concrete pavements containing units with as lean a mortar as used in these experiments, and when failures do appear, we are positive that such failures are due directly to the use of concrete having too great a slump or to the use of excess mixing water.

The fact appears to be overlooked that the durability of concrete is a factor of the density of the mortar used in the concrete and that as the density of the concrete decreases its absorptive power increases and the life of the cement is correspondingly decreased. The proof of these assertions is in freezing and thawing tests made by Professor Scholer on cores taken from pavements in New Jersey. These tests showed that the density of the mortar used in the concrete is the controlling factor.

*Harold P. Hayden**—The development of a standard laboratory test for the evaluation of methods for curing concrete is a complicated task and the authors deserve much credit for their efforts to this end. Unfortunately, however, there are several features of their technique which are open to serious criticism. It is essential in the development of a method of evaluation that the atmospheric conditions to which the test specimens are exposed be standardized in every detail. The principal factors involved in the rate of evaporation, other than the curing medium, are, in the probable order of their importance: (a) the relative humidity of the test atmosphere; (b) the velocity of air currents at the exposed surface of the specimen; (c) the temperature of the test; (d) the ratio of exposed surface to volume of specimen; (e) the gradation of particle size (including Portland Cement) occurring in the mixture, and, (f) the water-cement ratio.

It is suggested that data on velocity of air currents in the curing cabinet, as well as data on particle size distribution and the water-cement ratio of their standard mix, should be included by the authors in their paper. The conditions of high temperature, low humidity and high ratio of surface to volume of specimen were, no doubt, selected in order to magnify differences in the efficiency of the various curing methods. These extreme conditions have resulted in a critical method which is prone to give incongruous results. The meteorological conditions maintained in the curing cabinet would seldom occur in this country except in the arid regions of the Southwest. More concordant and practical results might have been obtained had the extremes of exposure conditions been modified.

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By resorting to the use of three layers of nine ounce burlap immersed in water at both edges and exposure in the presumably saturated atmosphere of an oven devoid of humidity control, the authors have contrived to keep the burlap wet in their experiments. Such methods are impossible in the field, yet, in case burlap is not kept wet while in contact with uncured concrete, it will act as a wick and accelerate evaporation of the mixing water. It is obviously futile to use data obtained under these experimental conditions with those obtained on specimens exposed at 32 per cent relative humidity in the evaluation of various methods of curing.

Best results with wet burlap curing were obtained by applying the treatment immediately after molding the specimen. This practice is impossible in highway construction as immediate application of burlap would mar the surface and obliterate the transverse brooming which is almost invariably used to produce a non-skid surface. Most burlap is applied at least an hour after finishing and in many cases the concrete will remain unprotected for three hours. In case burlap treatment is used as a preliminary to the application of a sealing material, a further period of unprotected exposure will occur between the treatments. An important feature of liquid curing materials is that they may be applied to the surface of freshly broomed concrete without danger of marring. Specifications for their use prescribe that they must be applied immediately. Unless concrete is treated immediately in hot dry weather, hair cracking or checking will inevitably develop in the surface.

It is difficult to understand, in the face of these generally known circumstances, why the authors chose to apply wet burlap immediately and defer the application of the liquid curing materials for three hours in most of their experiments. Reversal of this timing procedure would be more closely in accord with field practice. It is strange that there are no experiments reported in which the liquid seal was applied immediately after molding the specimen.

One to three hours exposure of an unprotected concrete surface at 100°F. and 32 per cent relative humidity is certain to cause surface dehydration with the probable formation of minute cracks. In the rather unusual practice of testing thin slabs of concrete in flexure, the authors have seen fit to place the upper surface of the specimen in tension by inversion. In view of the probability that the upper surface was seriously fissured by one to three hours exposure in the curing cabinet before the application of the liquid seal, it is not remarkable that the strength of specimens treated in this manner was found to be slightly lower than those which had received an immediate

preliminary treatment with wet burlap. Fissuring of the exposed surface will also explain the wide variation in values and many of the anomalies encountered in this series of tests. If the removal of the sealing compound from the surface of certain specimens by means of abrasion with a carborundum block was necessary to obtain complete resaturation, it would have been desirable to submit all specimens to a similar treatment. Abrasion would, no doubt, reduce the "skin strength" of the specimen as well as its thickness.

It is apparent that any of the liquid curing materials may be applied in a film of sufficient thickness to be substantially impervious. It will be granted that a truly impervious film will afford satisfactory curing. No statement is made in the paper as to the rate at which the various liquid materials were applied. Liquid curing material "A" is the most viscous product in this group and gave decidedly the best results. There is, therefore, reason to suspect that the relatively poor showing made by the other liquid seals is due to the use of thinner films than are specified by the manufacturers. It would be of interest to learn just how the proper application rate was determined.

In the last analysis, curing is ån economic problem. It is shown in Table No. 7 that three day burlap curing is slightly inferior to the best liquid seal and is definitely inferior to the combination of preliminary burlap followed by the better surface sealing materials. Immediate application of the liquid cures, at the rate specified, would undoubtedly have given better results than were reported.

As the preliminary use of burlap for three days will double the cost of curing, it is apparent that the contractor will prefer to use the liquid seals to the exclusion of burlap. The preference of the contractor for immediate application of a liquid curing material is amply proven by the growth of this method of curing during the last decade. More than fifty million square yards of pavement have been successfully cured with such Impervious Membranes. The use of these materials is also rapidly spreading into the field of structural and architectural concrete.

BY CARLL W. HUNT*

The authors are to be commended and deserve much credit for the work done and the presentation of the data developed in their investigation. However, in my opinion, the procedure used fails in its principal objective to formulate the laboratory procedure to determine effectiveness of various curing methods for the following reasons: (1) Procedure does not simulate actual job conditions; (2) Procedure

*Hunt Process Co., Los Angeles, Calif.—by letter.

permits comparison only between the individual curing qualities of various papers or liquid sealing compounds, but not a comparison between curing methods.

The writer, for more than 14 years, has endeavored to develop a laboratory test procedure that will itself permit exact determination of the comparative merits of various curing methods. He has yet to satisfy himself that it can be done without any question of the correctness of results. This is because it is impossible to simulate all job conditions in a laboratory, and consequently some of the data must be further investigated in the field or have field information already available for correlation with the laboratory findings. Generally speaking, laboratory procedure always results in water or paper curing specimens being cured in a much more efficient manner than prevails on a job, whereas the reverse is the case for specimens cured with liquid sealing compounds.

Referring specifically to the authors' procedure, it is noted that no reference was made to a free circulation of air over the specimens. Apparently such a condition did not exist, as the water lost during the first three hours of exposure without protection shows an average loss of less than five per cent. Specimens of the size used, exposed outdoors, would have lost a much larger quantity of water; probably as high as 20 per cent if the atmospheric conditions were similar to those employed in the laboratory procedure.

The specimens used had an exceptionally high proportion of area exposed in relation to the mass. Such a thin section of concrete would never exist on an actual job. This condition would not be reflected in the water cured specimens, but would affect the membrane cured, both paper and liquid.

Specimens consisted of a concrete composed of only cement and sand. I believe it is generally recognized that concrete composed only of a mortar will permit the loss of water much more readily than will a concrete containing a coarse aggregate. This condition would result the same as mentioned for the size of specimens, in that it would not effect the water cured but would the membrane cured specimens.

The water curing process employed in obtaining the ideal concrete cannot be duplicated in actual practice. It would be impossible to apply water in any form other than a very fine mist onto freshly placed concrete for several hours after the placing without damaging the surface texture or finish. The same deleterious effect would result from the early application of paper. On the other hand, liquid sealing compounds can be applied immediately after the subsidence of any

surface water, if any, or immediately after the finishing operations. Apparently this point was not taken into consideration by the authors. It is one of the principal advantages of the liquid sealing compound that the curing procedure can start several hours earlier than any other proper curing method without affecting the texture, and will prevent formation of the minute checks or fissures that develop in the concrete surface due to the rapid evaporation of the mixing water during the first few hours after placing.

Apparently no effort was made to determine the effectiveness of various rates of coverage of the liquid sealing compounds as no mention was made of this rate of application. It is another advantage of this type of compounds that the rates of coverage and the number of coats can be varied to meet the atmospheric conditions prevailing during the time of placing and curing of the concrete. The extreme conditions to which the specimens were subjected represent a condition that would prevail in this country only in the arid section of the southwest. They are, in some respects, even more severe in that the 100° F. temperature and the 32 per cent relative humidity remain unchanged for the entire duration of the curing periods. From the results, it would appear very logical that the liquid compounds were applied at the coverages suggested by the manufacturers for more normal temperatures, or possibly for temperatures and humidities universally used by most engineers in their laboratory investigations. Such temperatures are normally 70° and a humidity of from 50 per cent to 70 per cent. The temperatures and humidity conditions used in the procedure were most extreme and resulted in accelerated tests, which in the writer's experience have always given results that were far from uniform.

Relative to the procedure not resulting in data that would permit a proper comparison between methods and materials, it seems very definite that the following was the result:

1. The water curing procedure was perfect in every detail as to time of the first application of the water and the continuance of the condition of saturation. This procedure is impossible to follow on any actual construction project, no matter how determined and exacting the inspection and supervision may be. Already mentioned is the impossibility of starting the water curing procedure until the concrete has attained sufficient strength to prevent damage to the surface.

2. The paper curing procedure entailed the use of paper in perfect condition, application made immediately after forming of the specimens, or three hours later, and the existence of a perfect seal between

the edges of the paper and the container holding the specimen. This condition also is impossible on any job unless the paper is never re-used and constant supervision and sufficient labor made available to repair tears, reseal joints not originally properly sealed, and add additional weight to the paper wherever the wind finds an opening whereby it can get under the paper and lift it from the surface. This latter action occurs very generally and is particularly damaging as it creates "flue" and "bellows" action, thereby increasing evaporation very rapidly.

3. Specimens cured with the liquid sealing solutions apparently were covered with a definite quantity of the solution without particular regard to rate of coverage and time of application. The point has been previously mentioned as well as the effect of the size and type of specimen used.

The writer noted the authors' comment that the effectiveness of the liquid membrane coverings used was materially improved by applying the material three hours after molding as compared to application made one hour after molding. This condition could only exist if the surface of the concrete were so moist that the membrane resulting from the liquid formed before the moisture had subsided sufficiently to permit adhesion of the membrane to the concrete surface. This condition also might indicate incomplete compaction in the specimens when molded.

The writer believes generally the authors have interpreted their data very accurately, but that the conclusions drawn apply only to the peculiar condition imposed by their tests. It is his opinion that the reasons the investigations seemed to indicate the effectiveness of paper and liquid curing materials were increased after 24-hour application of wet burlap was because the original application, particularly insofar as the liquid curing materials were concerned, was delayed too long. Proper liquid sealing materials, without other than ordinary supervision, can be applied at the proper time and in sufficient quantity to insure the presence of more than sufficient mixing water to permit the complete hydration of the cement.

AUTHORS' CLOSURE

The authors agree with Mr. Hunt to the extent that the procedure used in their tests did not simulate all conditions to be found in actual job practice. It should be obvious that to simulate all possible job conditions in the laboratory would be an impossible task. Furthermore, it has been demonstrated that to conduct experiments of this nature in the field may give results entirely misleading. For instance,

several years ago we conducted an elaborate series of curing tests on a paving job in southwestern Tennessee. All of the accepted methods were employed and, in addition, a number of sections were given no curing treatment whatever. It so happened that rain fell on every one of the so-called "no cure" sections with the result that the tests compared very favorably with the sections cured with the best of accepted methods. This is but an illustration of the serious obstacles confronting an engineer carrying on curing experiments in the field. Such experiences formed the basis for the authors' contention that curing experiments should only be carried on in the laboratory under controlled conditions of humidity and temperature.

The authors do not accept Mr. Hunt's statement that the laboratory procedure resulted in water or paper cured specimens being cured in a much more efficient manner than would prevail on the job, and that the reverse was true for specimens cured with liquid sealing compounds. In this experiment the liquid curing materials were applied at a uniform rate and the amounts of coverage carefully weighed. It is felt that it would not be possible to apply liquid curing agents in the field with any greater care than was done in these tests. In fact it is extremely doubtful whether any field application would be as uniform as is possible in the laboratory.

Mr. Hunt is incorrect in assuming that the air in the curing cabinet above the specimens was not circulated. Not only was the air above the specimens circulated continuously, but in addition the air in the cabinet was changed periodically by an automatic air conditioning system.

Mr. Hunt feels that the use of thin sections of mortar in lieu of thick sections of concrete affected the membrane-cured specimens but not the water-cured specimens. However, no data are quoted or presented in support of this condition.

The text of the paper made it clear that the so-called "ideal" method of water curing used was for the purposes of comparison only. Otherwise, a different procedure would not have been used for the 1, 2, and 3-day burlap methods.

It is felt that the conditions of temperature and humidity employed in the test procedure are not as extreme as Mr. Hunt would have the reader believe. Unfortunately for the highway engineer a rise in temperature during the daytime is usually accompanied by a corresponding decrease in relative humidity so that the hottest hours of the day are the time when the lowest values for relative humidity are usually recorded. Accurate records taken just outside of Washing-

ton, D. C., indicated that during a typical construction season the relative humidity fell below 35 per cent on approximately 50 per cent of the total days covered which, of course, included days on which rain fell. Such data coming from a city which is generally conceded to be very humid are indeed interesting in view of the statements that an atmosphere of 32 per cent relative humidity is only found in arid regions. It should also be remembered that the temperature of 100° F. used in these tests was in the absence of sunlight, whereas the same cannot be said of the average road slab.

Mr. Abrams states that moisture is necessary for the continuing hydration of portland cement. The authors agree that this is true. In fact the data from the report indicate quite clearly that membrane coverings, which do not supply moisture to the concrete, are more effective when preceded by a preliminary curing of wet burlap. If all of the methods investigated supplied water to the concrete for continued hydration, it probably would have been unnecessary to conduct the investigation. It was because of the difference in ability of the various methods to supply water to the fresh concrete that the tests were made.

The authors do not claim that 100° F. and 32 relative humidity simulated actual field conditions although, as stated above, it is known that such climatic conditions are encountered during the average construction season in areas other than the semi-arid regions of the southwest. On the other hand, Mr. Abrams states that 75° F. and 60 per cent humidity simulates field conditions without offering data to show that these values are correct. In the authors' opinion such conditions are no more representative of the country as a whole than are the values which were used in the tests. Furthermore, it is a common practice in research work to subject the product under test to as severe conditions as will be encountered in service. Why should such a procedure be called an "accelerated" test?

Mr. Abrams lays considerable stress upon the fact that a perfect correlation was not obtained between water retention and strength. Perfect relationships are the exception rather than the rule in this type of research as anyone who has published as much data as Mr. Abrams should know. Again he stresses the fact that one complete series of tests did not give exactly the same relative results as another series using in both cases the same procedure. When it is considered that approximately two years elapsed between the time the first and last series were run, there is reason to believe that exact checks were not to be expected. The fact that the same *trend* in results was

obtained in the two series is the important thing. The fact that the numerical relations did not check exactly in every case did not, in the authors' opinion, affect the general conclusion in any way.

Mr. Abrams states that a 1:2 sand mortar will lose water much faster than will a standard concrete, the coarse aggregate running as large as 2½ inches. The authors recently concluded a short series of tests to throw light on this particular point. Comparison specimens of 1:2 mortar, similar to those used in the original investigation and of concrete containing coarse aggregate up to ¾ in., were prepared and cured under the original test conditions, that is at a temperature of 100° F. and a relative humidity of 32 per cent. An amount of water equal to 27 per cent of the total volume was used in the case of the mortar specimens as compared to 17 per cent of the total volume in the case of the concrete specimens. In spite of this difference in the original water content, the actual water losses from the surface of the specimens as determined daily for seven days were almost identical for the two mixes. This would indicate that the amount of water loss from the surface of concrete due to evaporation is primarily a function of the surface area and the atmospheric conditions above the surface rather than the type of mix used. The size of the specimens limited the maximum size of coarse aggregate which could be used in these tests. However, the authors feel that the two mixes were sufficiently different to indicate quite definitely that Mr. Abrams' point is not well taken.

Mr. Abrams in his discussion shows inconsistencies in reasoning. At one time he states that the thin specimens used would develop incipient cracking on the exposed surfaces, and that the "perfectly" cured specimens would be *least* affected because of the fact that an air seal was provided. At another time he criticizes the procedure because of the fact that the "perfectly" cured specimens did not give "perfect" strength. In fact he shows a table to prove that other methods gave strengths *higher* than the "perfect" method. The authors fail to see how these two lines of reasoning can be reconciled.

Mr. Hayden asks why a carborundum stone was used to remove the skin from some of the specimens and not others. All specimens were subject to the same treatment in this respect. This point was clearly brought out in the paper.

Mr. Hayden further asks how the proper rate of application was determined for the different curing materials. In every case the coverage used was in accordance with the recommendations of the manufacturer of the particular curing agent involved.

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Concrete Design vs. Concrete Placing—The Need for Cooperation*

By George L. Lucas†

SYNOPSIS

We are still suffering from the misconceptions which arose with heavily reinforced sections and sloppy concrete. We are continually confronted with plans and specifications which are not workable in practice. The problem, with our present knowledge of what good concrete should be falls on the construction forces and the inspection department. There should be coordination and cooperation at the outset in concrete construction among designers, specification writers and field engineers. It is too late for coordination or cooperation when the contract is let and the job is under way.

INTRODUCTION

In presenting this subject the intent is to avoid captious criticism, and merely point out certain conditions surrounding the manufacture of concrete, particularly reinforced concrete, which militate against the making of the best product. The purpose will be to reveal those predicaments arising from lack of cooperation and coordination among designing engineers, specification writers and construction engineers. No reproach is meant for any of the three, nor is the blame, if any, to be considered attaching to any one. Paraphrasing the modern writer's introduction to his novel, the characters in this presentation are entirely fictitious and bear no relation to any living person or group!

Since the field of concrete design and construction is so vast, and its ramifications innumerable, only a few simple typical cases will be cited to illustrate the problem, and perhaps point to a solution.

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WET OR DRY, RICH OR LEAN

In general, current malpractice is the result of past practice sanctified by age and usage. A few of you perhaps, like myself, can remember back some forty years or more, just prior to the Great Flood,—the days of antediluvian concrete, of concrete mixes so dry, harsh, unyielding and refractory that the concrete had to be laid carefully in thin layers and then thoroughly rammed until moisture—not water—appeared on the surface. Then came the Great Inundation, and for forty years we have been striving to stem the flood, and at last we are beginning to recover lost ground.

In the old days there were no difficulties in placing concrete because of the interference of steel. Concrete was tough, and arbitrary, and the steel, if any, had to be placed so you could get at the concrete and beat it into place. There was no misconception then as to how concrete should be placed. When the flood came, however, concrete was no longer placed—it was poured, because it was a fluid. The evils attending the excess of water were not confined to concrete, but extended to design. When the designing engineer conceived of concrete as a fluid, one that could be poured, many of the former exasperating limitations on his designing ability and technique were removed, and he could give full play to his genius and imagination.

Structural steel and steel bars could be placed any and everywhere, in any type of form, provided due regard was paid to the laws of hydrostatics in the design of the form! From the designing engineer's viewpoint, economy in design then reached its apex, particularly with respect to steel, for bars could be placed very closely together and interlaced, and structural steel pocketed within forms with a minimum clearance between the steel and form, and so on. The erroneous idea became fixed that the concrete mix *flowed* around this network of steel. It did not. It segregated. It honeycombed. And in tight places particularly between the flanges of beams or girders and their forms, large areas were never filled at all.

To this day we are continually confronted with plans of structures perfectly designed as to the economical distribution of steel, but practically impossible of construction with good concrete, because of the maze of complicated and closely placed reinforcing rods, and structural steel spaced with a minimum of clearance between forms, all preventing the placing of plastic concrete. A better appreciation of the practical handling or placing of plastic concrete on the part of the designer would go a long way toward ameliorating this condition.

It will be claimed, of course, that in accommodating the reinforcing steel to plastic concrete mixes, economy in steel is sacrificed. But is this true economy, if the steel is to be surrounded by weak and inferior concrete? Why sacrifice the concrete? Why not make each member in the structure sound, strong and durable?

In the design of structures and their members, the physical properties of the materials alone are considered, the principal one being strength—tensile and compressive. For concrete, practically compressive strength alone is considered. For steel, the section may be increased or decreased to meet exactly the requirements of the particular case. For concrete, however, the strength may be *safely* varied only within narrow limits. The designer, following the natural Scotch instinct of his species, will frequently economize on concrete, offering in justification the fact that 3,600 pound concrete, say, is unnecessarily strong, and 2,500 pound concrete is ample, the saving to be entirely at the expense of cement.

Let us inquire into the so-called economy or saving. A good *structural* concrete with a 4-in. slump and with a 28 day strength of 3,600 lb. is satisfactory for most structural requirements. The following comparison shows the difference between 3,600 and 2,500 concrete, which it is claimed is strong enough. The figures show the ingredients in the two mixes, and the material saving:

Cement, Bags/Cu. Yd.	Water, Gal.	Strength, 28 Days	Materials Cost
5.27	6¼	3600	$5.61/ cu. yd.
4.25	8½	2500	$4.95/cu. yd.
		1100 (30%)	$0.66

Now, with structural concrete in this neighborhood costing from $18.00 per cu. yd. upwards, in place, the materials saving—cement—is 3.7 per cent. The loss of strength is 30 per cent. And this is not the whole story. Along with the loss of strength occur other losses: decrease in density, increase in permeability, and reduced durability. The designing engineer should understand that strength is a very good measure of durability.

Now, I don't hold a brief for the cement industry. One may use too much cement. There is an optimum amount that may be used, however, for true economy.

THE SPECIFICATION WRITER

The average specification writer is seldom one who has had a complete and broad experience in design, in engineering materials, their

properties and manufacture, the manufacture of concrete and general construction in the field, yet he is required to draft the specifications covering in detail these broad subjects. In general he must be guided by contracts and specifications already in existence, and frequently in his innocence he believes the work they cover was carried out exactly according to these specifications. Sometimes he is unfortunate enough to have several such specifications at hand, where the clauses covering similar workmanship and materials are not identical, and he must choose. He is usually working under pressure. He seldom sees the contract drawings; is it any wonder that at times his specifications are not clear or particularly applicable to parts of the work?

Using the familiar vernacular, he has an "out," however. When he is up in the stratosphere and the gas gives out, he turns to Article Eighteen of the contract, and bails out with the parachute, reading as follows:

> These specifications require the furnishing and doing of all things necessary or proper for or incidental to the above construction. * * * All work shown on the contract drawings even though not expressly mentioned in these Specifications * * * and all things not specified either on the Contract Drawings or in the Specifications but involved in carrying out their intent and in the complete and proper execution of the Work are required by these Specifications; and the Contractor shall perform the same as though specifically delineated, described and mentioned. * * * The material and workmanship shall in every respect be in accordance with what, in the opinion of the Engineer * * * is the best modern practice. * * * The material and workmanship shall in every respect be in accordance with what, in the opinion of the Engineer * * * is the best modern practice of similar construction, and wherever the plans, drawings, specifications or other contract documents or the directions of the Engineer admit of doubt as to what is permissible and/or fail to note the quality of any work, that interpretation which requires the best quality of materials and workmanship in conformance with the best modern practice of similar construction must be followed.

Let us look at a specification for concrete that was actually written, containing the following provisions, here somewhat abbreviated:

> Coarse aggregate shall be 1½ inch or where permitted ¾ inch broken stone.
>
> The slump shall be between two and three inches.
>
> The concrete shall be poured.
>
> Spouting or chuting of the concrete will be permitted under certain conditions. If chuting be permitted the slope of the chute shall not be steeper than one vertical to two horizontal.
>
> Excess water and laitance on top of the concrete shall be removed by siphons or by pumping.
>
> Concrete shall be rodded, spaded, and/or vibrated.

As an interesting experiment, make a concrete mix with 1½ inch broken stone coarse aggregate, having a slump of two to three inches.

Try pouring it! Try chuting it on a one to two slope chute. Try chuting it at almost any slope! Good concrete cannot be chuted.

The contractor, reading this specification with its permissible methods of handling the concrete, immediately makes up his mind to use chutes, and bids accordingly.He is actuated by the profit motive. He would like to give you two to three inch slump concrete, but he could prove from the first batch that it would not go down the chute. With low slump concrete, one hardly expects laitance and water on top, but ample provision has been made for removing it in large quantities. He is prepared and willing to do this, because he is fully equipped.

The difficult situation, you see, has now been transferred to the field engineer and the contractor for solution, under the terms of the before mentioned parachute Article Eighteen! The contractor is familiar with this article which appears in all contracts, and this familiarity has bred something approaching contempt for it. The result for the owner is not so happy.

CONCRETE INSPECTION

We will now wander over to the site. * * * For good concrete there are required not only the proper ingredients, but first-class intelligent inspection at the mixer and at the point of deposit. It is not uncommon to find in a field organization the younger, and consequently less experienced, men used as inspectors; or the old and old-fashioned type of inspector, who has been an inspector on masonry or concrete for ages—the one who knows he knows concrete, and who in spite of your instructions will produce his type of concrete.

With the great development of commercial central mixing plants miles away from the site of the work, and the delivery of the concrete in truck mixers, the closest inspection is required at the central mixing plant and at the job, and there must be close and quick telephone communication between the job inspector or engineer and the mixing and batcher plant inspector. In spite of the best design and the best materials, the best specifications will not produce good concrete without competent supervision by intelligent and experienced inspectors. There are many doubtful practices in the field which I shall avoid mentioning, assuming that with ample and competent engineering and inspection suprevision they are easily corrected.

HURRY

One of the principal causes of defective or inferior concrete is hurry, the dictionary definition of which is "confused haste." Good concrete requires very careful, unhurried handling. Its quality depends on

careful manipulation, step by step, and if any one step or if all the steps be hurried, the concrete will resent it and will react, if not immediately, certainly all too soon.

Contracts and specifications which do not provide ample time for properly making and placing concrete are responsible for much of our defective work, and many of our structures suffer from this cause. The time frequently is so limited that the contractor must resort to doubtful expedients to get the work out, and the construction engineer must acquiesce. The fault, however, lies not with the construction engineer, because it is his duty to have the work finished in time. It s generally caused by the demands of Capital: "Hurry! Hurry! Finish! The interest on the investment is running!" Invested capital hates paying interest. It would rather receive dividends.

It is high time that our chief engineers took a firm stand against such time limitations, which are reflected too often in the workmanship in our structures.

Such haste often results in unsightly looking structures, because it prompts the making of placements of too great size. An instance of this type of difficulty is shown in the following example of a rush job. The concrete for the topping of a tower base on a bridge was deposited in approximately twenty hours. The size of this pier was 34 feet x 34 feet in plan and 24 feet high. This roughly is a thousand cubic yards of concrete. When the forms were stripped it was found that surface cracks had developed in the concrete, due to uneven cooling of this rapidly deposited mass. It is undoubtedly not the best practice to make continuous placements of such size; however, it was necessary to do so to meet the construction schedule.

The concrete for the second tower pier topping was required to be placed in the same time. In an effort to reduce the number of shrinkage cracks in the second pier, a blend of sulphate resistant cement and natural cement in the ratio of five bags of portland cement and one bag of natural cement, was used. Experiments made in the laboratory showed that in the cement that was to be used this expedient for overcoming the difficulties caused by haste did reduce the heat of hydration at early ages. However, with other cements this mixture had practically no effect in reducing the heat of hydration at early ages. When the forms were stripped on the second tower pier the number of shrinkage cracks was found to be greatly reduced, but a few still appeared. When construction programs are planned, consideration should be given to the extent of each lift, since no way is known of changing the materials so that no harmful results will occur when the manufacture and placing of the concrete is unduly hurried.

The accompanying illustrations show the effect of cooperation and the lack of such cooperation between designing and field engineers.

Fig. 1, 2, and 3 show a job where cooperation prevailed. The concrete was of 3 in. slump; coarse aggregate 1½ in. vibrated. The pictures show the proper placement of reinforcement bars in rigid frame structures, leaving ample space between the bars not only for

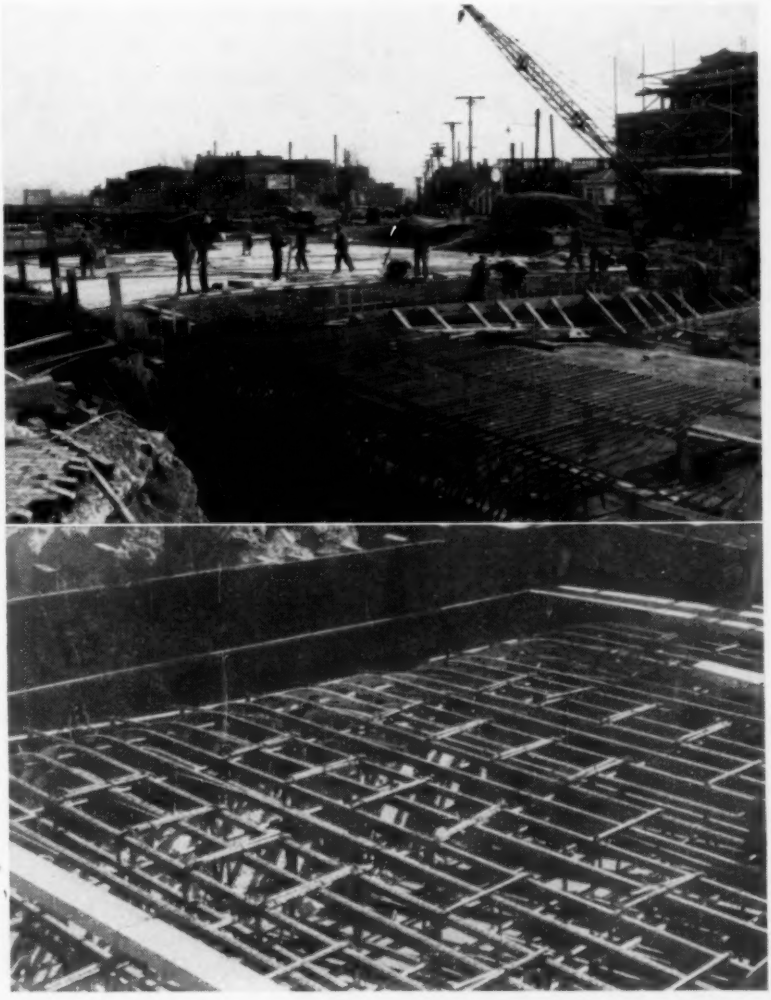

FIG. 1 AND 2—AN INSTANCE OF CO-OPERATION, FURTHER ILLUSTRATED IN FIG. 3

the placing of plastic concrete but for the introduction of an internal vibrator. In Fig. 1 the bars are so spaced that when the additional bars are placed among them (Fig. 2 and 3) there is still room for the placing of plastic concrete and the introduction of the vibrator. Bars in these structures are in the same vertical plane, as far as possible. Too often in this type of structure the bars are placed so closely that they screen out the coarse aggregate; or else there is required a concrete of much greater slump and with less strength, density and durability.

The effect of cooperation is shown again in Fig. 4 where concrete of 4 to 5 in. slump was to be placed; coarse aggregate ¾ in.; vibrated. The conditions might result in a very difficult placement for good concrete. The work is the ceiling of a vehicular tunnel where the glass tile encased in bronze grippers for the ceiling finish are laid on forms over which are placed the reinforcing rods. The reinforcing rods, it will be noted, are placed above one another in the same plane, and there is space enough between for the introduction of an internal vibrator. The concrete was pumped into place. The headroom for working varied from zero at the edges of the ceiling to about six feet in the middle.

Fig. 5, cooperation once more. The concrete of 4 to 5 in. slump; coarse aggregate ⅜ in. and ¾ in.; vibrated. This too is a ceiling in a vehicular tunnel, where further complications are encountered because of exhaust and fresh air flues, requiring wire mesh around the beams and also between the beams at flue locations. In this case 4 in. of concrete, using pea gravel as coarse aggregate, was used, and immediately on top of that, the normal concrete with ¾ in. coarse aggregate, all vibrated in place, making a monolithic slab.

Fig. 6 shows the conditions for concrete of 3 in. slump; coarse aggregate 1½ in.; vibrated—a perfectly natural mistake in underestimating the screening effect of reinforcement. The original design called for heavy girder beams with about 12 in. clearance between flanges, the beams acting with the concrete to form a girder between two piers, over which was placed about 30,000 cu. yds. of concrete in the anchorage. The concrete in the anchorage was required to be placed in a very short time and work was continuous, night and day, until completion. At the request of the contractor, railroad rails spaced to allow 3 in. clear between flanges were permitted as a substitute for rolled girder beams. The rails acted as a screen and clogged up immediately. It became necessary to reduce the size of the coarse aggregate to ¾ in., and increase the slump, in order to place the con-

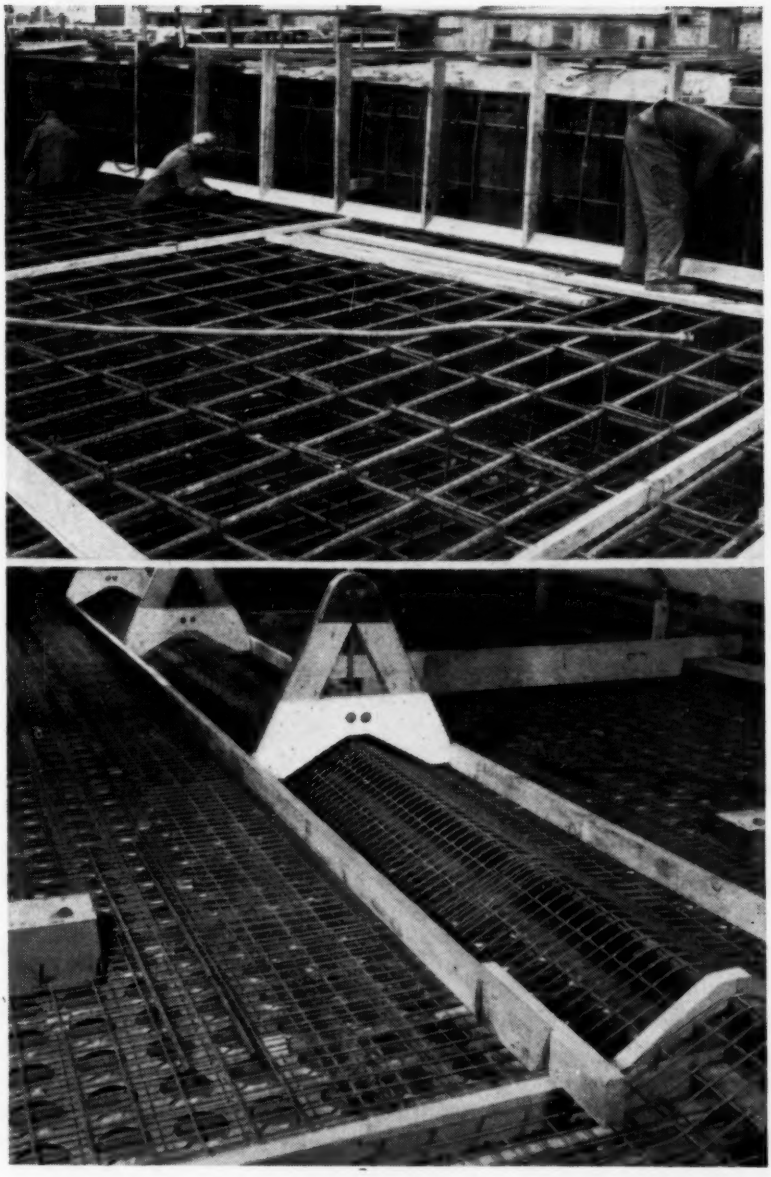

FIG. 3—STEEL PLACED WITH A VIEW TO PLACING CONCRETE (SEE ALSO FIG. 1 AND 2)

FIG. 4—THIS WORK MIGHT HAVE BEEN DIFFERENT EXCEPT FOR THE FORETHOUGHT IN THE DISPOSITION OF STEEL

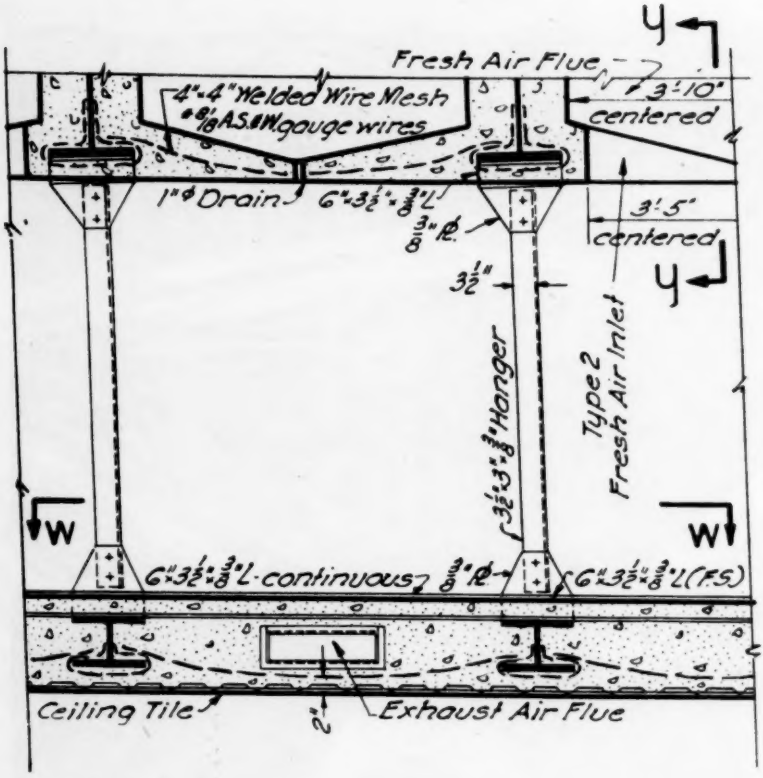

FIG. 5—THIS TOO WAS A DIFFICULT PLACING JOB WHERE CO-OPERATION WAS WORTH WHILE

crete. In the area where the two layers of rails lapped over the bottom tiers further difficulties were encountered.

Fig. 7 shows lack of cooperation. Concrete of 3 in. slump; coarse aggregate, first design ⅜ in., second design ⅝ in., final design ¾ in.; vibrated. A pre-fabricated bridge floor was made in sections approximately 5 x 50 ft. It was originally designed with the intention of having only ½ in. of concrete over the steel, it being the idea of the designing engineer that ⅜ in. broken stone could be used and would make a normal concrete. It was demonstrated that ⅜ in. broken stone as coarse aggregate in concrete was equivalent to mortar with all its defects, including excessive shrinkage, and ⅝ in. broken stone coarse aggregate was suggested. Laboratory trial batches of the ⅝ in. coarse aggregate, however, showed this size impracticable because of the great excess of cement required to give the proper

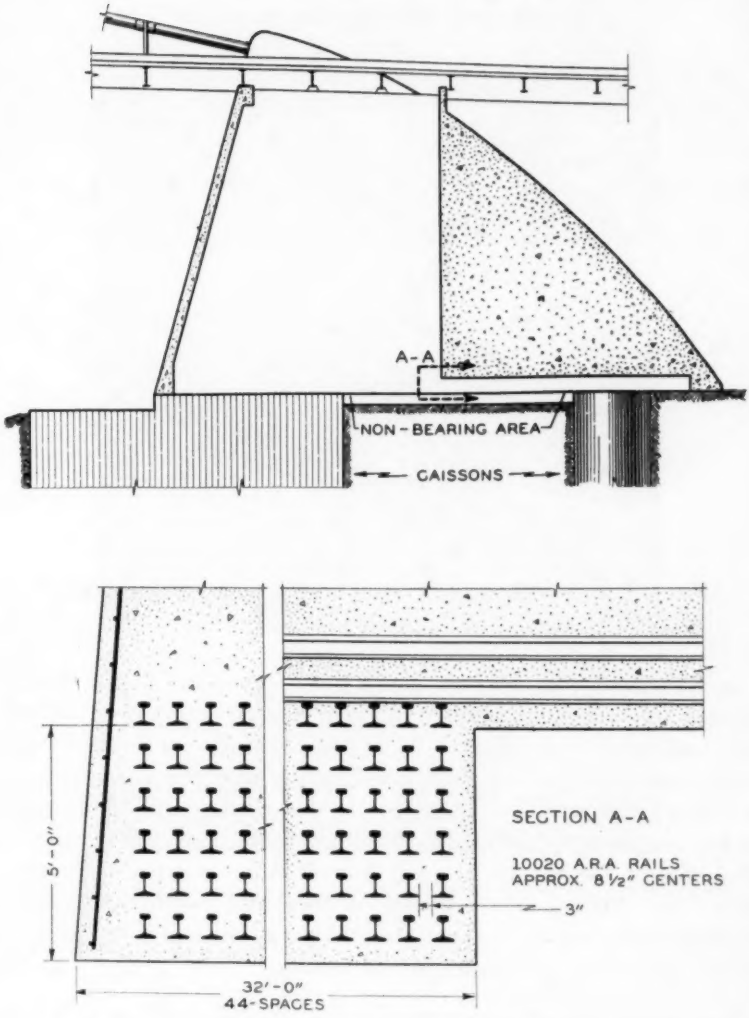

FIG. 6—THIS ARRANGEMENT CAUSED TROUBLE IN PLACING AND THE MIX HAD TO BE CHANGED

strength and wearing properties, and because the shrinkage in such concrete was 60 per cent greater than for normal concrete with larger stone. Eventually the design permitted a ¾ in. cover over the steel, and ¾ in. broken stone aggregate was used successfully. Unless designers are thoroughly familiar with the radically different characteristics of ⅜, ⅝, and ¾ in. coarse aggregates when used in concrete serious mistakes are likely to be made in the resulting concrete.

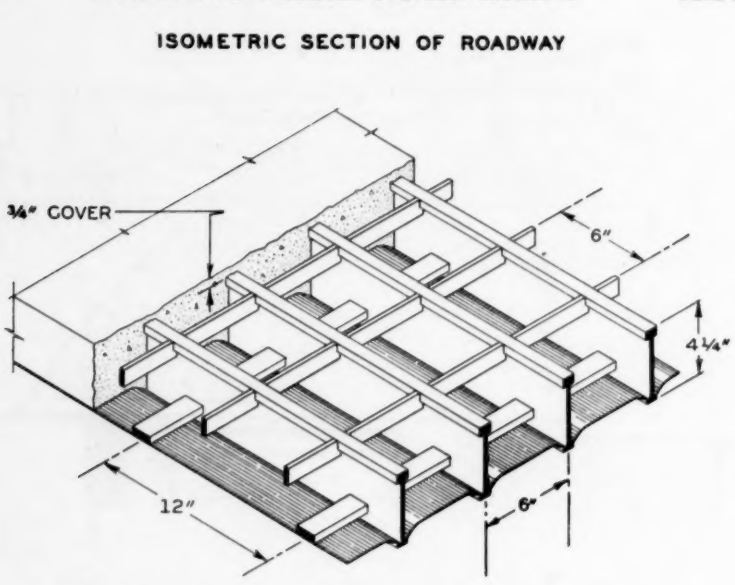

FIG. 7—THE DESIGN DID NOT LEND ITSELF TO EXECUTION

Fig. 8 shows a job in which there was cooperation and forethought in design. Concrete of 3 in. slump; coarse aggregate 1½ in.; vibrated. These hollow piers average 90 ft. in height, and were designed originally as smooth shafts from top to bottom, with the additional requirement that they be rubbed smooth. It was pointed out that construction joints would show in spite of the rubbing, and V joints were suggested, fixing the height of each lift of concrete with the result as shown. No rubbing was required on the piers, no form or construction joints were visible, as they were identical with the V joints. The piers as they stand are structures of beauty.

There was a lack of cooperation in the work shown in Fig. 9. Concrete 3 in. slump; coarse aggregate 1½ in.; vibrated. The structures are similar in some respects to those in Fig. 8. Provision was not made for definitely fixing the lifts and marking them with V joints. The concrete is excellent concrete, and the same in all respects as that shown in Fig. 8, except for the construction joint marks. While this concrete was rubbed, the construction joints are still visible.

Fig. 10—Again lack of cooperation. Concrete 3 in. slump; coarse aggregate 1½ in.; vibrated. Structure, similar to those in Fig. 8 and 9 but no requirement for marking off construction joints with V

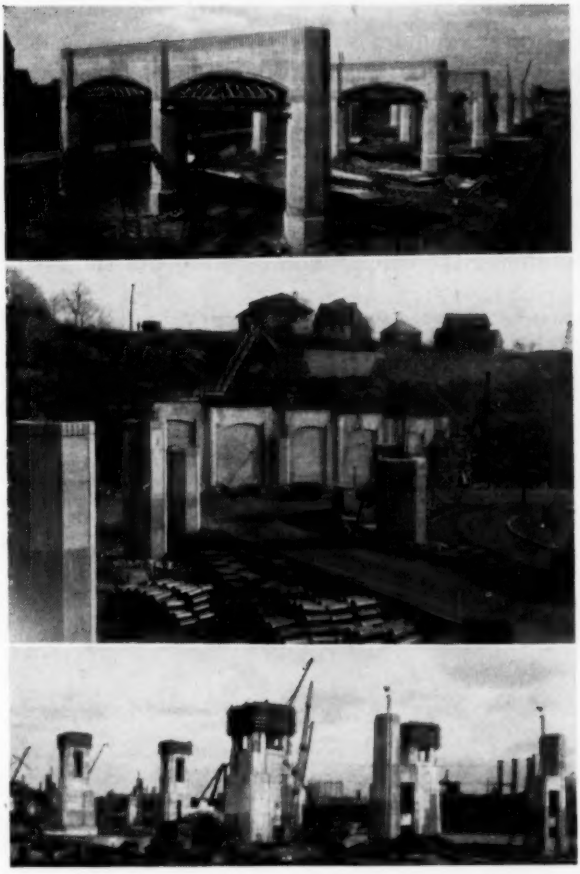

FIG. 8—THE V-JOINTS MAKE A VIRTUE OF NECESSARY CONSTRUCTION JOINTS

FIG. 9—A JOB SIMILAR TO THAT IN FIG. 8 WHERE THE CONSTRUCTION JOINTS ARE OFFENSIVE

FIG. 10—THE V-JOINT AS NOT USED HERE EITHER

joints, with the results shown. Each lift of concrete is plainly visible and marked off by a construction joint which does not disappear on rubbing.

CONCLUSION

From the foregoing sketchy though impartial review, it must be plain that the difficulties arise from lack of coordination and cooperation among the three principal groups—designers, specification writers and field engineers. In general the drawings of the structure are nearing completion before the specification writer begins his work.

The field engineer is engaged in his own line of activity in the field, and is not in direct contact with either the designing department or the specification writer. At the time the specifications are being written, the great rush begins to complete the drawings and print and advertise the contract, at which time it is a little too late for getting together.

The solution is a very simple one. Full knowledge of all the details of the contract do not lie in any one group, and a little time spent in consultation among the three would cure most of the existing evils.

Discussion, to close in the September Supplement, should reach the A. C. I. Secretary by July 20.

Vol. 35 PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

(Supplement)

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *SEPTEMBER 1939*

Discussion of a paper by George L. Lucas:

Concrete Design vs. Concrete Placing—The Need of Cooperation*

DISCUSSION BY FRANK E. KULAS AND THE AUTHOR

BY FRANK E. KULAS†

Mr. Lucas' paper is a particularly valuable contribution to the concrete profession at this time. The vast public works programs presently under way throughout this country as well as the prevailing ideal concrete weather suggest that large volumes of concrete are being placed. A lesson in cooperation of concrete placing and concrete design is indeed a timely one and worthy of consideration.

The paragraph entitled "concrete inspection" was of especial interest to the writer—however, he feels that the importance of that subject should have warranted more details. He thus offers some suggestions or additions.

The inspection of concrete is essentially the last important step of concrete control. It may embrace errors or corrections depending upon the cooperation with the specification writer or the designer of the mix. In the lack of that cooperation, the responsibility of proper concrete rests with the inspector—which in turn—depends upon his experienced judgment. As the author stated in his article that the experience of an inspector is often questionable, the need of cooperation of the specification writer and the mix designer should further be encouraged for the mutual understanding of proper concrete control.

It is advisable that the specification writer interpret or explain fully to the inspector before the start of each new job the actual enforce-

*JOURNAL, Amer. Concrete Inst., June 1939; *Proceedings* Vol. 35, p. 501.
†Pittsburgh, Pa.

ment of the specifications—those parts where the inspection should be more intensified than others. It would relieve the inspector from several unnecessary arguments arising from undue and unjust enforcement of minor or unwarranted details. It would also tend to weave diplomatic and trustworthy relations with the contractor. In short, good inspection is referred to as good judgment. And good judgment—according to a law maker—which in this case is the specification writer—is sound engineering interpretation of the specifications.

As to the mix designer, the writer suggests that mixes be designed in accordance with the general method of conveyance to be adopted. As an instance, in most of the municipal jobs of to-day, ready-mixed concrete by truck transit from batching plants is selected. The capacity of these trucks, most commonly, is of 2½ cu. yd. volume. The mix designer generally specifies the cement quantity on a one cu. yd. basis—say 5½ bags per yd. or in some odd weight say 530 lb. On a 2½ cu. yd. computation, the 5½ bag mix requires 13¾ bags or 1293 lb. of cement, and the 530-lb. mix in a 2½ yd. batch is 1325 lb. or 14.09 bags. If the batching plant has ample facilities for weighing either of the suggested quantities, no question is raised. However, because of the several brands of cement on the market, and since each brand is usually specified to be tested before its use, the problem of weighing different brands of cement for the different jobs becomes an economic one. Hence the measurement by bags is resorted to. And because of this factor—the fractions of the bags are not placed to the computed accuracy—regardless of the closest inspection.

The writer thus suggests that a mix designer specifies his mix quantity in even bags for the particular volume capacity unit to be adopted in transportation—unless the placing volume is sufficiently large enough to justify the weighing of the cement of one brand throughout the job. Referring to these computations, the writer would suggest that the 5½ bag per yd. mix be set at 14 bags for 2½ cu. yd. capacity, and the 530 lb. mix per yd. be also set at 14 bags for the 2½ cu. yds.

A question perhaps may arise that the water-cement ratio is altered in setting an even bag 2½ cu. yd. mix. Computations show on the 5½ bag mix that the difference is approximately .02, and the 530 lb. mix difference is slightly less than .01. Such slight differences may show but negligible strength results.

Another question that may be raised is the price difference because of the altered cement quantity. That could be amicably settled—especially with the batching plant owner. His cost of cement handling will result less.

In conclusion, such details corrected by the cooperative understanding between the inspector, the mix designer, and the specification writer would tend to establish good concrete practice.

AUTHOR'S CLOSURE

The author appreciates Mr. Kulas' constructive comments concerning inspection of concrete manufacture. Care was taken in the paper not to emphasize inspection, because proper treatment of this subject alone would require a paper of considerable length, defeating the purpose, which was to emphasize the necessity for cooperation between the several groups having to do with designing, specification writing, construction and inspection.

Perhaps a very brief outline of the methods of concrete inspection on the works with which the author is connected may be helpful in covering some of the details Mr. Kulas has in mind. These works are those of The Port of New York Authority, the Triborough Bridge Authority and the New York City Tunnel Authority, and include the Lincoln Tunnel, Queens Midtown Tunnel, Triborough Bridge, Bronx-Whitestone Bridge, and George Washington Bridge improvements.

All concrete for these works is controlled from a central point, the Port Authority laboratory. It is a completely equipped and manned chemical and physical laboratory for the inspection and testing of engineering materials used in construction. There is also attached a large force for field inspection which inspects cement at mills; sand, gravel, broken stone at pits and quarries; and inspects batcher and mixing plants, etc.

The "water cement ratio" method is specified for the design and control of concrete. The cement is paid for as a separate item by the owners. The ingredients are weighed, although bags of cement may be used as units of weight. Flexibility in weighing methods is permitted, provided the methods are precise.

The laboratory force calibrates all batcher plants before approval, and checks them from time to time. Mixer plants also are inspected before approval. The several types of mixer trucks are carefully inspected, and rules issued for the use of each type.

Inspectors reporting to the laboratory are placed at batcher plants, and mixer plants, and are in constant contact with the laboratory and field. When variations or changes are required in the mix, they are made immediately. The proportions by weight of ingredients used from time to time or within the same day are changed according to the fineness modulus of the aggregates being used and the moisture

content. The concrete is tested daily by a testing force from the laboratory equipped for making specimens, for storing, curing and removal to the laboratory.

While the above is necessarily exceedingly brief, nevertheless it should indicate that for the proper inspection of concrete and its ingredients there is no point, from the beginning of the manufacture of the ingredients to the mixing and placing of concrete and its curing where inspection can be safely slighted.

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Some Factors Influencing Results of Pull-Out Bond Tests*

By Carl A. Menzel†
MEMBER AMERICAN CONCRETE INSTITUTE

SYNOPSIS

This paper presents data from studies made at the Research Laboratory of the Portland Cement Association, designed to show the effect of some of the many factors influencing the results of pull-out bond tests. It represents a progress report in an extensive investigation of bond resistance. Data presented cover the influence of the following factors on the loads developed in pull-out specimens; type of bar surface, length of embedment, type of lug on deformed bars, position of lug, consistency of concrete, thickness of concrete around the bar, position of the bar with respect to the placing of the concrete, leakage of water from the molds immediately after placing. Results are given for concrete of different cement contents and ages at time of test. Load-slip curves are presented based on measurements at both the loaded and the free end of the bar. Comparison is given between data from the present investigation and those obtained in earlier studies.

INTRODUCTION

In reinforced concrete a comparatively small volume of steel is embedded in a large volume of concrete. Since steel is a much stronger and tougher material than concrete it is possible for a small volume of properly embedded steel to carry a large proportion of the load imposed on the concrete encasing the steel. The transfer of load or stress from the concrete to the steel is made possible by the resistance established between the concrete and surface of the embedded steel bar. The higher this resistance to relative motion or slippage under stress the more effective will be the desired interaction between the concrete and the steel. This resistance to slippage has been called

*Presented at the 35th Annual Convention American Concrete Institute, New York, Mar. 1-3, 1939.
†Associate Engineer, Research Laboratory, Portland Cement Assn., Chicago.

bond or bond resistance. Without any bond resistance the embedded steel would be practically useless. With inadequate bond only a fraction of the desired interaction between concrete and steel would be obtained resulting in waste of material or an unsafe assembly. With adequate bond the primary requirement for proper interaction is met and both steel and concrete can be stressed as intended by the designer with economy and safety. It is apparent, therefore, that definite information on the subject of bond resistance is necessary and of great practical importance. It is also apparent that since it involves interaction between two such widely different materials as concrete and steel, each of which is subject to the influence of many variables, many combinations of variables are involved which make a proper study of bond resistance complicated and extensive.

Bond was intensively studied, by means of pull-out and beam specimens, at the University of Illinois by Abrams between 1909 and 1912, resulting in Bulletin 71* comprising 238 pages, published in 1913. Notwithstanding this extensive research, Abrams found it advisable in 1919 to conduct further research on bond with pull-out tests at the Structural Materials Research Laboratory at Lewis Institute, which was reported in Bulletin 17†, published in 1925. In general, the results of these two researches provided the principal source of information on bond in relation to practical design problems. However, in recent years there has gradually developed among concrete technicians and designers a feeling that information on bond resistance is too incomplete to answer many questions of common interest in connection with modern concrete.

Some of these questions have been answered by an excellent series of papers by Messrs. Gilkey, Chamberlin and Beal, and by recent papers by Messrs. Davis and Kelly, Withey, Wernisch, Shank, Mylrea and others. However, so many factors are involved and so many different combinations encountered in the assembly of steel and concrete that many important questions will probably remain unanswered for years to come.

The purpose of this paper is to show the effect of a few of the factors influencing the results of pull-out bond tests.

GENERAL DESCRIPTION

Each pull-out specimen consisted of a single bar embedded in a concrete prism. Some prisms were cast with the bars horizontal as in beams; in others with the bars vertical as in columns. In the

*Tests of Bond between Concrete and Steel, by D. A. Abrams.
†Studies of Bond between Concrete and Steel, by D. A. Abrams.

majority of specimens the bars were horizontal. Usually the thickness of concrete surrounding a bar corresponded to the minimum required by the A. C. I. or joint code. All prism specimens were cast in metal molds *with bars rigidly held in the position desired.* The molds were sealed against leakage and so arranged that the various parts could be disassembled at 20 hours without damaging or straining the concrete and embedded bar. The detail for accomplishing this without permitting leakage where the bar passed through the ends of the mold is shown in Fig. 1a, Appendix.

All bars were either 1-in. round or square free from loose rust and scale and were carefully cleaned with naptha so as to be free from oil or grease. Special care was used in proportioning the concrete. Most of the test specimens were made with relatively lean concrete containing 5 sacks of cement per cu. yd., although in some tests concrete containing 4.3, 6.0 and 6.8 sacks per cu. yd. was used. Tests were made with both 2 in. and 6 in. slumps. All cement consisted of a laboratory mixture of 4 brands of normal strength cement with a surface area of approximately 1800 sq. cm. per gram. All aggregate was Elgin sand and gravel with size range 0-¾ or 0-1 in. depending on the clearance available between bar and the sides or bottom of the specimen. The maximum size of aggregate was not more than ⅔ of the minimum clearance. In all cases the concrete was mixed dry ½ min. and wet for 2 min. in a Lancirick open tub type mixer. The concrete was allowed to stand an additional 3 or 4 min. while the slump test was being made. Usually the concrete was placed in the first mold within 7 minutes after the beginning of mixing. All concrete was placed and compacted with the utmost care by hand rodding.

All tests reported herein were on damp specimens, moist cured either for 7 or 28 days. Further details relative to the proportions of the mix and the gradation of the aggregate are given in the appendix.

Fig. 1 shows several views of a typical pull-out prism under load and details the location and method of mounting the various .0001-in. dial gages for measuring slips at the free (top) and loaded (bottom) ends of bar. Further pertinent remarks on the test set-up and slip measurements follow:

Slip at the loaded end is measured by the lower 2 gages fastened to a steel frame, clamped firmly to the concrete with a clearance 1/16-in. above the base place. Each gage point contacts the smooth surface at the end of a horizontal eye-bar which is held around the pull-out bar by means of three 5/16-in. set screws with cupped ends. The horizontal bar is free to move in a suitable recess provided in the rigid base plate. Before starting the test, the horizontal bar is carefully

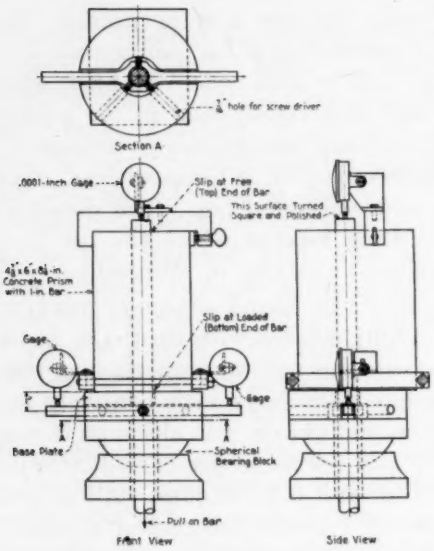


FIG. 1—DIAGRAM OF PULL-OUT SPECIMEN UNDER LOAD SHOWING POSITION OF .0001-IN. GAGES FOR MEASURING SLIPS AT FREE (TOP) AND LOADED (BOTTOM) ENDS OF BAR

See text—"General Description."

fastened to the pull-out bar at a distance of 1 in. below the bottom surface of the concrete. As load is applied the gages indicate the movement of the point on the reinforcing bar at which the horizontal bar is attached with respect to the point on the concrete at which the gages are attached. The amount of this movement is determined by the algebraic average of the gage readings at each load. Since it was impossible to attach the gages and the cross bar at points exactly at the face of the specimen it was necessary to subtract from the movement indicated, the elongation of the bar in the 1-in. distance between the point of attachment and the face of the specimen. This elongation is determined by the stress in the bar due to the load and the known modulus of elasticity. Strictly, there should have been subtracted from the movement also the deformation in the short length of concrete between the face of the specimen and the point of attachment ($\frac{5}{16}$-in.). This movement, however, was so slight that this correction was neglected.

The slip at the loaded end of the bar referred to in this paper and indicated on certain of the figures is this net movement. Ignoring the correction due to the compression of the concrete mentioned above,

this net movement represents an actual movement of a point on the steel originally at the face of the specimen, with respect to the base plate. At low loads this movement consists of two elements: (1) The elongation of the steel within the concrete due to the stress which will vary from a maximum at the face to zero at some point within the specimen; (2) The distortion of the concrete over the same length. At higher loads it includes also any movement due to the slip of the free end of the bar. The shape of the load slip curves, constructed on this basis, shows that movement began at the lowest loads which it was possible to read accurately. This movement continued at an increasing rate with increased load and showed no tendency to sudden change of rate when slip at the free end of the bar began. After the steel stress exceeds the yield point, the readings of bottom slip become unreliable because of unknown allowance for bar stretch and tendency for eye bar and pull-out bar to loosen. The steel frame to which the lower gages are fastened is loosened or removed as soon as readings are discontinued.

Slip readings at the free end of deformed bars were usually made until the maximum resistance was developed either by splitting of the concrete or at excessive stretch of the steel bar. In a few cases the gage was removed just before failure to avoid damage from violent shock at failure at high loads. With the hydraulic type of testing machine this shock effect was materially less than with the older lever type. With plain bars load and slip readings were generally continued well beyond the point of maximum resistance.

The various types of 1-in. plain and deformed bars represented by the tests reported herein are illustrated by the views of Fig. 2. Further details regarding the bars, the steel from which they were made, and the nature of the bar surface are given in the notes which accompany this figure.

DISCUSSION OF RESULTS

Special Notes—A feature of these tests which should be emphasized is that in all cases the concrete was placed and compacted with the utmost care by hand rodding. The results therefore probably represent greater uniformity and a higher level of values than would normally be expected under conditions which prevail in concrete construction. This procedure was followed as the one best calculated to eliminate variability due to workmanship.

Early in the work of this investigation and in the study of data from other sources, it was found more convenient and significant to express the resistance developed in the pull-out tests in terms of steel stress

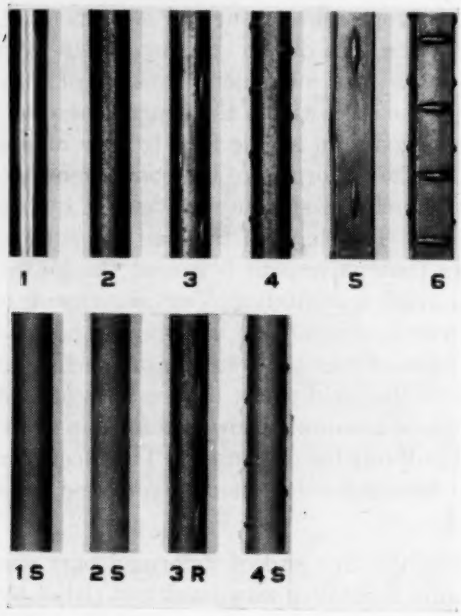


FIG. 2—VIEWS OF TYPES OF PLAIN AND DEFORMED BARS TESTED

BARS SHOWN ARE 1-IN. DIAMETER ROUNDS OR SQUARES AS FOLLOWS:

No.	Type of Steel	Yield Point	Ultimate
1	Round cold rolled, ground and polished accuracy stock	62,000	80,000
2	Round plain hot rolled bar, mild steel	32,000	57,000
3	Round deformed bar of rail steel with longitudinal lugs and smooth surface	50,000	80,000
4	Round deformed bar of intermediate grade steel with transverse lugs on 2 opposite sides	40,000	70,000–90,000
5	Square deformed bar of rail steel with longitudinal lugs	50,000	80,000
6	Square deformed bar of intermediate grade steel with transverse lugs on each of 4 sides	40,000	70,000–90,000

No. 1S, 2S and 4S represent bars No. 1, 2 and 4 respectively after bar surface was altered by sandblasting.

No. 3-R represents type of bar as No. 3 except that the bar surface (as delivered) was pitted and substantially rougher than the smooth surface of No. 3.

in the bar rather than in terms of the average bond stress per square inch of surface area of bar. For this reason all the data in this report, with the exception of Fig. 8 and 17, are plotted in terms of the pull on the bar expressed in lb. per sq. in. of cross-section.

Space does not permit the citation of all the excellent papers in the literature on bond. A selected bibliography is included in the paper, The Bond between Concrete and Steel, by H. J. Gilkey, S. J. Chamberlin, and R. W. Beal* and a more comprehensive list in the paper on Use of High Elastic Limit Steel as Reinforcement for Concrete, by H. J. Gilkey and G. C. Ernst†.

*JOURNAL, Amer. Concrete Inst., Sept., 1938; *Proceedings* Vol. 35, p. 1.
†Proc. Highway Research Board, Vol. 14, 1935, p. 271.

To conserve space and the time of the reader, the test data are presented graphically in the various figures without tabulations. There are many other factors affecting bond resistance, but those discussed in connection with Fig. 3 to 17 below comprise a convenient grouping for reporting results at this stage of the study. The salient features of the test data are discussed with respect to the several figures.

Effect of Surface of Bar on Resistance Developed at 7 Days. Each diagram in Fig. 3 shows that an increase in embedded length results in an increase in steel stress or pull required to produce the various slips indicated at free end of bar. The increase was especially marked for the deformed bar represented (No. 4 and 4S).

Bar No. 1 with ground and polished surface developed the lowest steel stress. The maximum stress shown occurred without preliminary slip as in the case of other bars but was accompanied immediately by a long slip suggesting very low running frictional resistance to movement of bar. Second loadings (results not shown in diagrams) developed markedly lower resistances than in first loading, indicating that static friction was also very low.

When the polished surface of Bar No. 1 was altered by sand-blasting to produce Bar No. 1S the steel stress developed at different lengths of embedment was about 4 times that of the original bar with polished surface. In this case the maximum stress occurred after a small preliminary slip.

The sand-blasting of the plain round hot rolled bar (No. 2) to produce Bar No. 2S also resulted in a marked improvement in steel stress at various slips and maximum. Even the sand-blasting of the surface of the deformed bar No. 4 improved the resistance developed by Bar 4S at all slips shown.

The sand-blasted hot rolled bar (2S) was definitely superior to the sand-blasted cold rolled bar (1S) due to such factors as an initially pitted surface, and more irregularity in diameter, roundness and straightness of bar.

Effect of Surface of Bar on Resistance Developed at 28 Days. The diagrams in Fig. 4 present test results on the same type of bars and specimens reported in Fig. 3 except that the specimens were moist-cured 28 days instead of 7 days.

In general the remarks made in connection with Fig. 3 also apply to this figure. As may be expected the resistances developed with the longer curing are appreciably higher but the relative difference from diagram to diagram for the different bars remains unchanged.

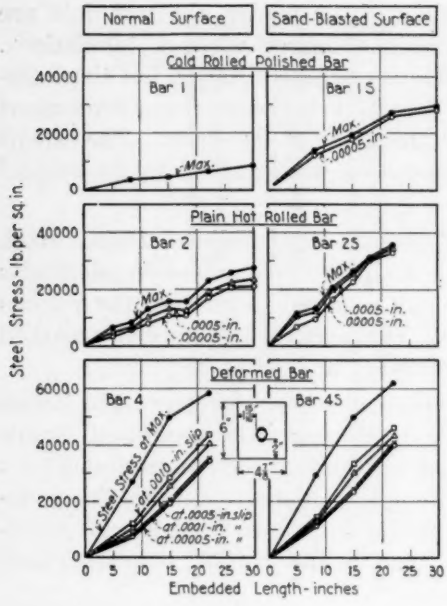


FIG. 3—EFFECT OF SURFACE OF BAR ON RESISTANCE DEVELOPED AT 7 DAYS

All bars 1 in. diameter embedded horizontally in prisms of cross section shown in lower diagram, and of variable length. Concrete: 5.0 sacks cement per cu. yd.; 6-in. slump (W/C — 0.64 by weight.) Specimens moist cured 7 days; tested damp. Slips indicated are at free end of bar. Values for each length of embedment are based on test of a single specimen.

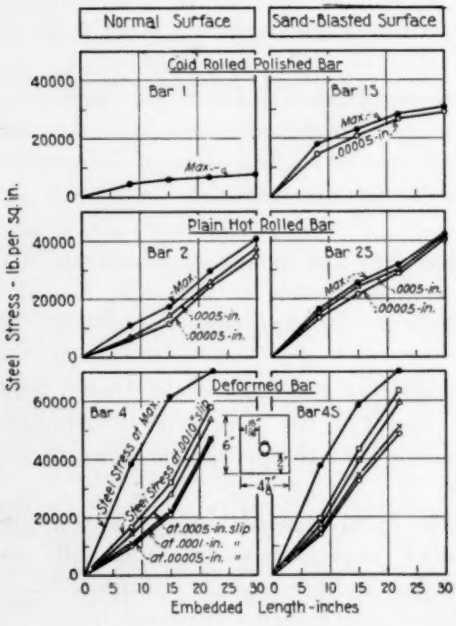


FIG. 4—EFFECT OF SURFACE OF BAR ON RESISTANCE DEVELOPED AT 28 DAYS

All bars 1 in. diameter embedded horizontally in prisms of cross section shown in lower diagram, and of variable length. Concrete: 5.0 sacks cement per cu. yd.; 6-in slump (W/C — 0.64 by weight). Specimens moist cured 28 days; tested damp. Slips indicated are at free end of bar. Values for each length of embedment are based on test of a single specimen.

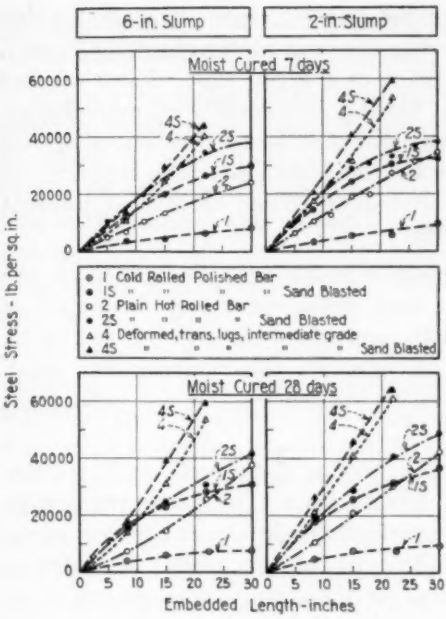

6-in. Slump
2-in. Slump
Moist Cured 7 days
Moist Cured 28 days
Steel Stress - lb. per sq. in.
Embedded Length - inches
1 Cold Rolled Polished Bar
1S " " " " Sand Blasted
2 Plain Hot Rolled Bar
2S " " " " Sand Blasted
4 Deformed, trans. lugs, intermediate grade
4S " " " " " Sand Blasted

FIG. 5—EFFECT OF SURFACE OF BAR AND CONSISTENCY OF CONCRETE ON RESISTANCE DEVELOPED AT SLIP OF .0005 IN, AT 7 AND 28 DAYS

The points show in the diagrams at the left (for 6-in. slump) are identical with those shown in the six diagrams of Fig. 3 and 4 for resistances developed at slip of .0005 in. at free end of bar. The diagrams at the right show similar points for 2-in. slump (water-cement ratio 0.59) not shown in Fig. 3 and 4. Other details as indicated in the notes for Fig. 3 and 4.

It is of interest to point out that the plain round hot rolled bar (No. 2) when embedded 30 in. (30 diameters) developed a steel stress of 37,000 lb. per sq. in. at .0005 in. slip and 41,000 lb. per sq. in. at maximum which was reached at a slip of about 0.003 in. These resistances are from 5000 to 9000 lb. higher than the yield point (32,000 lb. per sq. in.)

Similarly interpolating from the curves for deformed bar No. 4 the steel stress for an embedded length of 20 in. (20 diameters) would be about 46,000 lb. at .0005 in. slip and 67,000 lb. at maximum which would be limited by splitting of the prism. These resistances are well above the yield point of 40,000 lb. per sq. in. and close to the ultimate of 70,000-90,000 lb. for the intermediate grade steel used.

The alteration of the bar surface by sand-blasting showed a definite effect on the resistance developed with all types of bars plain or deformed. With the deformed bar (4 and 4S) the maximum resistance (terminated by splitting) was not altered by sand blasting but the resistances developed at various slips was substantially higher for the sand-blasted bars at all embedments. These tests as well as some of the later ones represented in Fig. 6, 7, 8, 9 and 10 strongly indicate that the nature of the bar surface is an important factor whether the bars are plain or deformed.

Another important point brought out by these diagrams, which should be kept in mind in reviewing the data plotted in other figures, is that the resistance characteristics of a given type of bar are not accurately indicated by simply testing one length of embedment. Likewise it is inconclusive to compare the bond resistances of two different types of bar from tests on the same embedded lengths. It appears that curves similar to those shown based on several different lengths of embedments are necessary.

Attention is also called to another result well illustrated by the curves for Bars 4 and 4S. The steel stress which was developed at maximum increased sharply with embedded length up to 15 in. (L/D = 15). For the longer embedments this increase was much less showing the effect of pronounced stretch beyond the yield point in causing splitting.

Effect of Surface of Bar and Consistency of Concrete on Resistance Developed at Slip of .0005 in. at 7 and 28 Days. The six curves in each diagram in Fig. 5 reveal at a glance the large differences in steel stress at a .0005-in. slip developed by the types of bars tested for different lengths of embedment. The curves for the plain bars in general show a gradual increase in steel stress with embedded length but at a diminishing rate. The deformed bars (4 and 4S) show the largest increases in steel stress in relation to the embedded length. The rate of change seems to increase slightly with the length of embedment. A similar trend is indicated by the curves for the hot rolled bars, (2) for both 6 and 2-in. slumps when moist-cured 28 days.

The effect of the consistency of the concrete is shown by comparing the curves for the 6 and 2-in. slumps which show that the stiffer mix developed the highest steel stress regardless of type or surface of bar or time of curing. This improvement in steel stress, due to the use of the stiffer mix, can only be expected when the concrete is placed very carefully around the bar by hand rodding as with these specimens, or by some types of vibration.

Effect of Type of Lug and Surface of Bar on Resistance Developed at 7 Days. In Fig. 6 the graph for square bar No. 6 with transverse lugs shows that substantially higher resistance was developed at all values of slip at the free end of bar and at maximum than by square bar No. 5 with longitudinal lugs. Similarly round bar No. 4 with transverse lugs showed resistances markedly superior to round bar No. 3 with longitudinal lugs.

This superiority with transverse lugs is important, since it holds even though the yield point of the bars with the transverse lugs was

less than that for the bars with the longitudinal lugs, (40,000 lb. for bars 4 and 6 and 50,000 lb. for bars 3 and 5).

The curves indicate that about 20 in. (20 diameters) at slip of .0005 in. would be sufficient to develop a steel stress of about 42,000 lb. (2000 lb. above the yield point) for both square and round bars (4 and 6) which had transverse lugs. On the other hand, it is noted that with this slip and embedment only about 20,000 lb. steel stress would be developed with the square and round bars (3 and 5) with the longitudinal lugs, which is far below the yield point of rail steel (50,000 lb.).

The maximum resistance (developed at splitting of the concrete prisms in both cases) was also markedly higher for bars of transverse lugs than for longitudinal lugs. It should also be pointed out that the maximum resistance shown for bars 3 and 5 with longitudinal lugs was only developed after very large slips; much larger than could be tolerated in reinforced concrete structures. Hence the maximum resistances shown for bars 3 and 5 cannot be realized and can hardly be considered in the same light as similar values obtained with the intermediate bars. Further detailed information on this item will be given in Fig. 8.

Reference to Fig. 2 shows that bar No. 3R had a noticeably rougher surface texture than bar No. 3 of the identical design. Bar No. 3R represented the only bar in the entire investigation of commercial run of bars purchased in the open market which had a surface as rough as shown. All other bars of this design had surfaces as smooth as indicated by bar No. 3.

The marked influence of this rough surface on the resistance developed will be apparent in comparing diagrams 3 and 3R in Fig. 6. For embedments of 20 in. the resistances are roughly 50 per cent higher at all slips and at maximum with the rough surface than with the smooth surface. Nevertheless the composite diagram in the lower right corner indicates that at slip of .0005 in. the resistance of bars 4 and 6 with transverse lugs and smooth surface was markedly higher than for bar 3R with longitudinal lugs and very rough surface.

Effect of Type of Lug and Surface of Bar on Resistance Developed at 28 Days. The diagrams in Fig. 7 for tests at 28 days confirm the relative superiority of the bars with transverse lugs shown in Fig. 6 for tests at 7 days. For example, at slip of .0005 in. and with embedded lengths of 20 in., bars No. 3 and 5 with longitudinal lugs show less than half of the resistance developed by bars No. 4 and 6 with transverse lugs.

Bar No. 3R with rough surface developed about 50 per cent higher resistance than bar No. 3 with smooth surface, again showing the importance of the character of the surface.

Influence of Type of Lug and Surface of Bar on Maximum Pull and Slip Developed at Splitting of Concrete Prisms. The lower diagrams of Fig. 8 clearly show that the total pull that must be exerted on a bar to split a concrete prism of the same type is markedly influenced by the type of lug and surface of the bar. In the extreme left lower diagram the pull at splitting is plotted for the 5 bars, 3, 5, 3R, 4 and 6 arranged in ascending order of pull. The same order was followed in

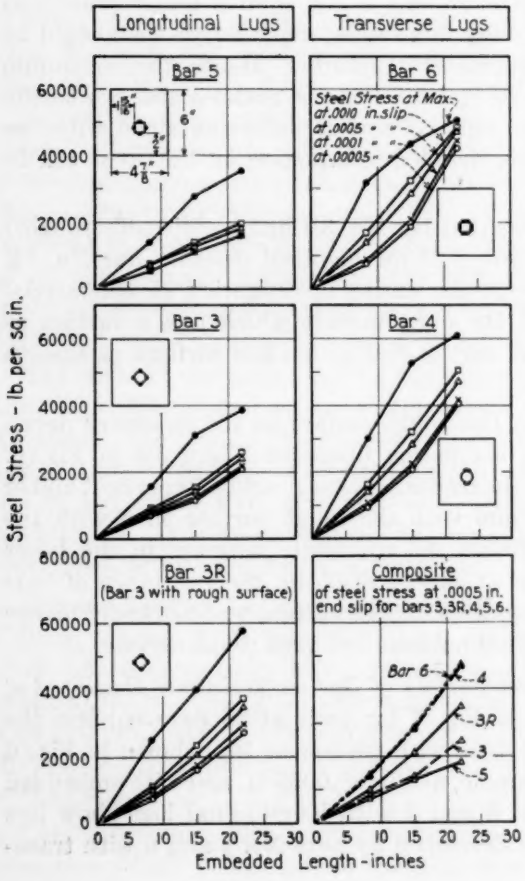


FIG. 6—EFFECT OF TYPE OF LUG AND SURFACE OF BAR ON RESISTANCE DEVELOPED AT 7 DAYS

The bars were 1 in. round or square as indicated. For further details see Fig. 2. All bars were embedded horizontally in prisms of the same cross-sectional dimensions as detailed in diagram for Bar 5. Values shown for each length of embedment represent the average resistance developed by concrete of 2 and 6-in. slump (1 specimen each) with cement content of 5.0 sacks per cu. yd. (W/C — 0.59 and 0.64). Specimens moist cured 7 days; tested damp. Slips indicated are at free end of bar.

plotting the values for the different bars in all other diagrams of this figure including the upper diagrams by which slip instead of pull is featured.

The lower diagrams all show that regardless of embedded length, slump of concrete or duration of moist curing, bars No. 3 and 5 with longitudinal lugs split the concrete prism with a pull of approximately 40 to 60 per cent as great as that required with bars No. 4 and 6 with transverse lugs. However, bar No. 3R showed that when the bar surface was rough instead of smooth the pull required to split the concrete was markedly higher especially with 22-in. embedments than for bars with longitudinal lugs and smooth surface.

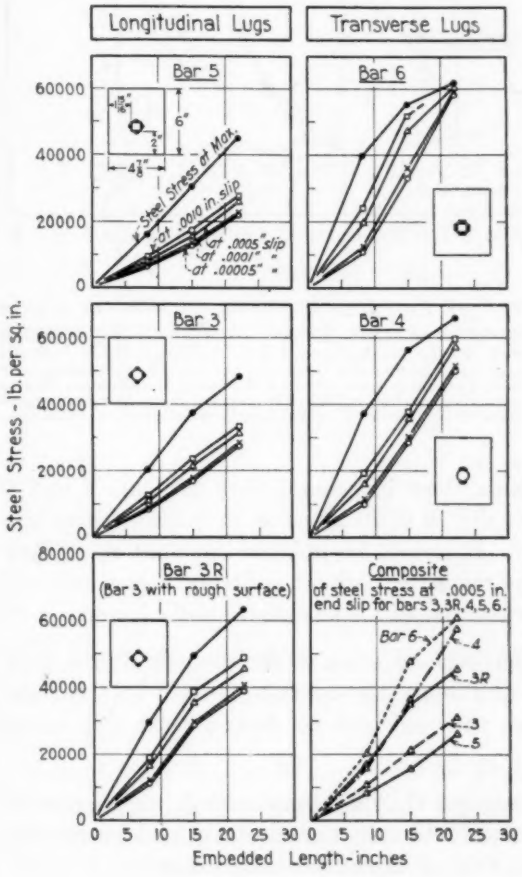


FIG. 7—EFFECT OF TYPE OF LUG AND SURFACE OF BAR ON RESISTANCE DEVELOPED AT 28 DAYS

The bars were 1 in. round or square as indicated. For furthur details see Fig. 2. All bars were embedded horizontally in prisms of the same cross-sectional dimensions as detailed in diagram for Bar 5. Values shown for each length of embedment represent the average resistance developed by concrete of 2 and 6-in. slump (1 specimen each) with cement content of 5.0 sacks per cu. yd. (W/C = 0.59 and 0.64). Specimens moist cured 28 days; tested damp. Slips indicated are at free end of bar.

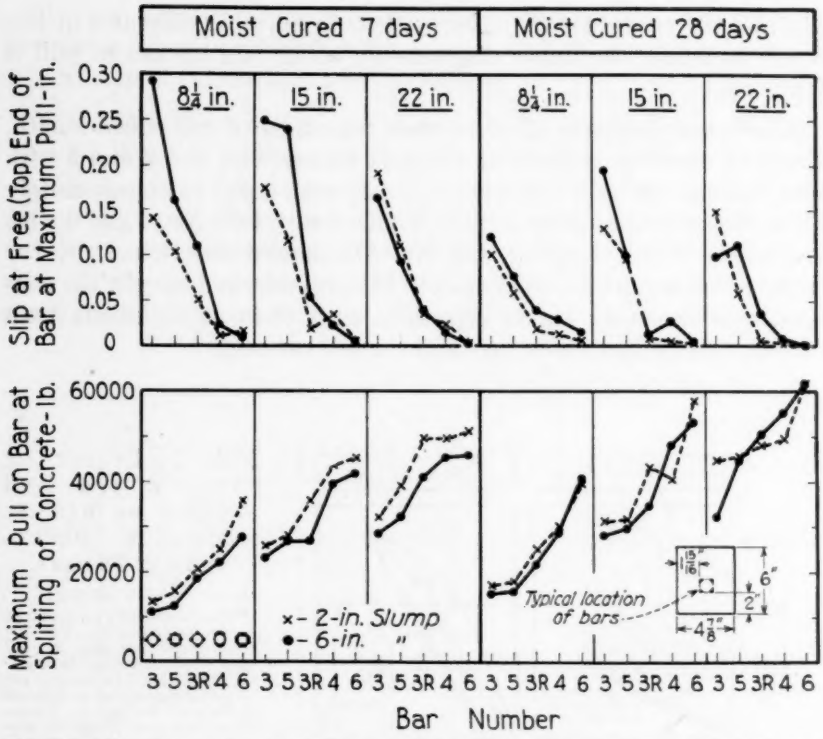


FIG. 8—INFLUENCE OF TYPE OF LUG AND SURFACE OF BAR ON MAXIMUM PULL AND SLIP DEVELOPED AT SPLITTING OF CONCRETE PRISMS

Bars 1 in. round or square as indicated. For further details see Fig. 2. All bars embedded horizontally in prisms of cross-section shown in lower right corner. Concrete: 5.0 sacks cement per cu. yd.; slump 2 and 6 in. (W/C = 0.59 and 0.64). Specimens moist cured 7 and 28 days; tested damp. Points shown in the upper and lower diagrams for any given bar, embedded length, and slump represent respectively the slip at the free end and the total pull on the bar when splitting of the concrete occurred (one specimen of a kind).

The upper set of diagrams show in general that the bars 3 and 5 with longitudinal lugs must slip 10 to 20 times or more than transverse bars to split the specimen. It should be pointed out that these long slips with longitudinal lugs result in pulls at splitting about one-half that for transverse bars where the slip is so substantially lower.

Bar No. 3R with rough surface showed far less slip at the load which produced splitting, and hence the overall effect of this surface roughness was to increase the pull and to decrease the slip when splitting occurred.

In general it will be observed that bars embedded in concrete of 2-in. slump required higher pulls to split the specimen and lower slips than were developed in the 6-in. slump concrete specimens.

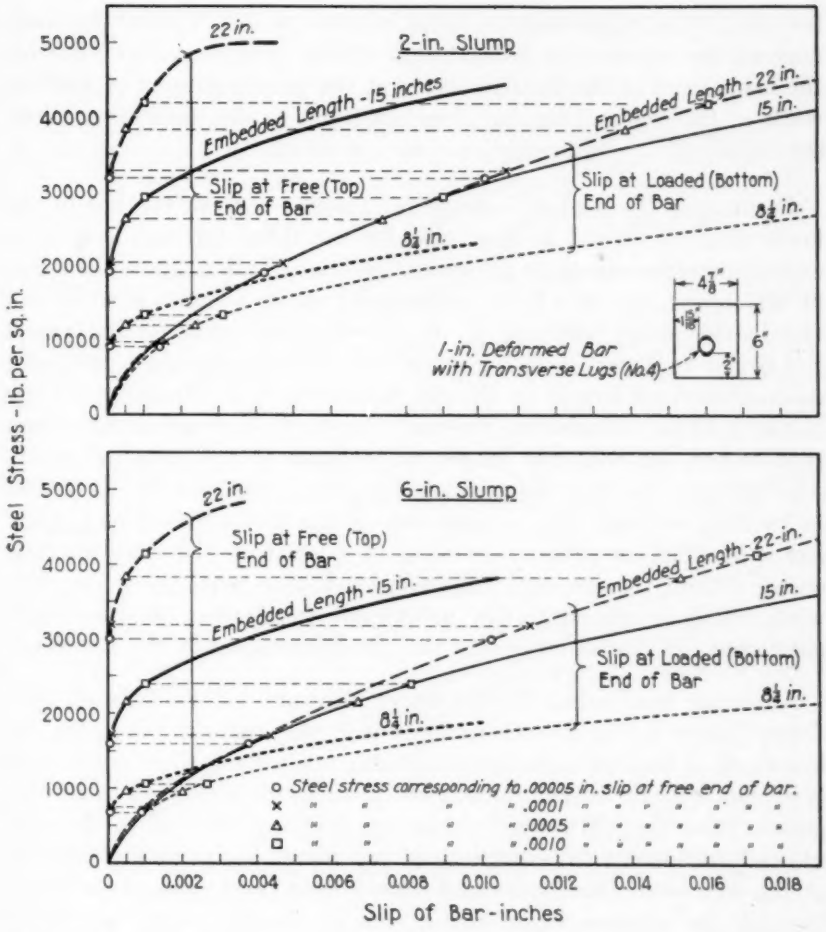


FIG. 9—LOAD SLIP CURVES FOR 1-IN. ROUND DEFORMED BAR WITH TRANSVERSE LUGS—SLIPS MEASURED AT BOTH FREE AND LOADED ENDS OF BAR

All bars (Type 4) were embedded horizontally in prisms of the cross-section as shown in upper diagram. Concrete: 5.0 sacks cement per cu. yd.; 2 and 6-in. slumps (W/C = 0.59 and 0.64). Specimens moist cured for 7 days; tested damp. Each curve based on average of 2 similar specimens. See text describing Fig. 1 under General Description for method of making slip measurements at loaded and free ends of bar.

Load Slip Curves for 1-in Round Deformed Bar with Transverse Lugs—Slips Measured at Both Free and Loaded Ends of Bar. The diagrams of Fig. 9 show the results of measurements of slip at the loaded end of the bar with simultaneous measurements of slip at the

free end. The upper diagram is for concrete of 2-in. slump; the lower diagram for concrete of 6-in. slump. These diagrams show that for the loaded end of the bar slip occurs at the very beginning of loading, while at the free end the bar does not begin to slip until a very considerable pull (stress in steel) has been reached.

Comparing the load-slip curves for the two ends of the bar in the lower diagram, it will be seen that for the three different lengths of embedment, the curves for the loaded ends are identical up to the stress at which the bar of 8¼-in. embedment shows a slip at the free end (steel stress about 7000 p.s.i.). At this stress the curve for the loaded end begins to depart from the other two, since the slip from this point on is determined largely by the slip at the free end. Similarly, for the bar with 15-in. embedment, the curve for the loaded end departs from that of the bar with the longer embedment at the stress at which slip begins at the free end (steel stress about 16000 p.s.i.). It should be pointed out that what is here termed slip at the loaded end of the bar is in effect an actual movement of a point on the steel where it emerges from the concrete with respect to the particles of concrete with which it was originally in contact (neglecting, as previously mentioned, the small correction due to deformation in the concrete).

A similar comparison in the upper diagram for concrete of 2-in. slump, shows a like behavior except that the load-slip curve for the lower end of the bar with 15-in. embedment does not show any departure from that for longer embedment until a pull is reached somewhat greater than that at which the free end first appears to slip. For the 8¼-in. embedment the departure of the curves begins at the first slip at the free end. The horizontal dotted lines in the diagrams drawn through the stresses representing slips at the free ends of .00005, .0001, .0005, and .001 in. make it easy to compare the corresponding slips at the loaded ends.

The shape of the load-slip curves for slip at the loaded end of the bar indicates that the slip increases at a more rapid rate than the steel stress for all lengths of embedment. Taking for convenience the curve for 22-in. embedment in the lower diagram for 6-in. slump, it will be seen that an increase in steel stress from 20,000 to 30,000 lb. or 50 per cent doubled the slip.

Load-Slip Curves for 6 Types of 1-in. Bars—Slips Measured at Both Free and Loaded Ends of Bar. The diagrams of Fig. 10 compare the resistance characteristics of bars of different types at 22-in. embedment.

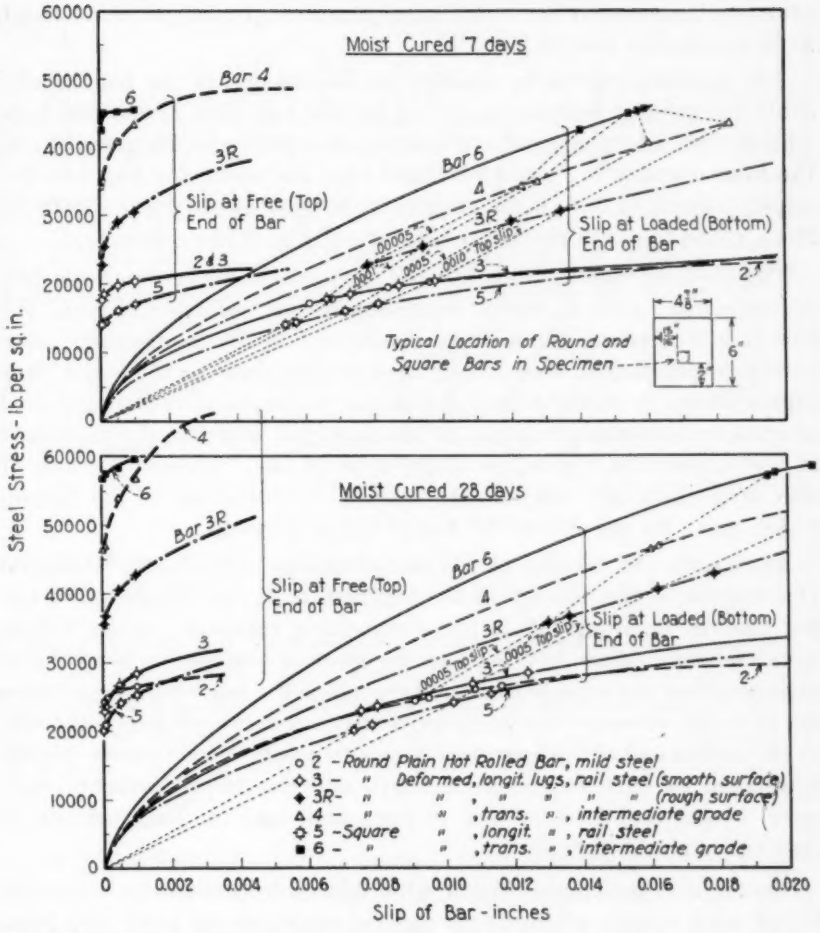

Moist Cured 7 days
Bar 4
Bar 6
Bar 3R
Slip at Free (Top) End of Bar
Slip at Loaded (Bottom) End of Bar
Typical Location of Round and Square Bars in Specimen
Moist Cured 28 days
Bar 3R
Steel Stress - lb. per sq. in.
Slip of Bar - inches
2 - Round Plain Hot Rolled Bar, mild steel
3 - " Deformed, longit. lugs, rail steel (smooth surface)
3R- " " " " " " " (rough surface)
4 - " " , trans. " , intermediate grade
5 - Square " , longit. " , rail steel
6 - " " , trans. " , intermediate grade

FIG. 10—LOAD-SLIP CURVES FOR 6 TYPES OF 1-IN. BARS—SLIPS MEASURED AT BOTH FREE AND LOADED ENDS OF BAR

The 6 types of bars tested are identified in the lower diagram, for further details see Fig. 2. All bars embedded horizontally 22 in. in prisms of cross section shown in the upper diagram. Concrete: 5.0 sacks cement per cu. yd.; 6-in. slump (W/C = 0.64); moist cured for 7 or 28 days and tested damp. Each pair of load-slip curves for each bar is based on a single specimen except for bar 4 which is based on 2 specimens. See text describing Fig. 1 under General Description for method of making slip measurements on loaded and free ends of bar.

The load-slip curves for slips at the free end of the bar again reveal the marked superiority of the bars with transverse lugs (bars 4 and 6). They indicate also the small difference in resistance developed by the round or square bars with longitudinal lugs (bars 3 and 5) and the

ordinary plain round hot rolled bars (2) unless the bar surface is rough as in the special case of bar 3R.

The load-slip curves for slips at the loaded end of the bar likewise show the greater resistance offered by the bar with transverse lugs. This is shown by the greater stress required to produce a given slip at the loaded end. It should be noted that the curve for bar 4 in the upper diagram (7 days moist curing) is the same as the curve shown for 22-in. embedment in the lower diagram of Fig. 9 (6-in. slump).

The inclined dotted lines in both upper and lower diagrams join points on the several curves representing the indicated slips at the free ends of the bars. These lines ar labeled ".00005 in. top slip", etc. It will be noted that when these lines are projected to the origin they represent nearly straight lines indicating proportionality between load or steel stress (at a given slip at the free end) and the slip measured at the loaded end, regardless of the type of bar. The explanation of this proportionality will be seen from a consideration of the factors which make up the slip of the bar at the loaded end.

The amount of the slip at the loaded end as defined under General Description, is due mainly to the summation of the elongation of the bar and the deformations in the surrounding concrete. Since, within working limits, these deformations are more or less elastic, it would be expected that the total slip at the free end of the bar would be proportional to the stress in the bar. Obviously, that proportion of the slip at the loaded end of the bar which is due to compression in the concrete is affected by such factors as strength and elastic properties of concrete, length and cross-section of specimen, and for long periods of loading by plastic flow.

Similar companion specimens with 2-in. slump concrete were also tested with results of the same general character as have just been brought out.

Effect of Thickness of Concrete under and at the Sides of 1-in. Round Deformed Bar on Resistance Developed. The tests upon which the curves in Fig. 11 are based were made to determine the influence of depth of coverage on the resistance developed in pull-out specimens. For the 8¼-in. embedment, and for a given grade of concrete, the loads at end slips of .00005 to .0010 in. were nearly the same for the three depths of coverage. This was also true for the 15-in. embedment in the leaner concrete (4.3 sacks per cu. yd.) but for the 15-in. embedment in the other two grades of concrete and for the 22-in. embedment in all grades, the stresses developed at a given end slip varied with the different depths of coverage.

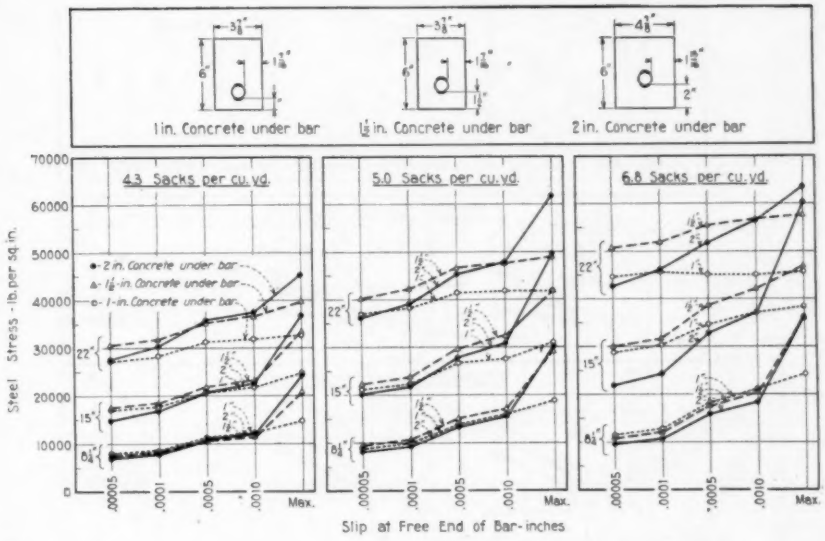

1 in. Concrete under bar
1½ in. Concrete under bar
2 in. Concrete under bar
4.3 Sacks per cu. yd.
5.0 Sacks per cu. yd.
6.8 Sacks per cu. yd.
Steel Stress - lb. per sq. in.
2 in. Concrete under bar
1½-in. Concrete under bar
1-in. Concrete under bar
Slip at Free End of Bar-inches
Max.

FIG. 11—EFFECT OF THICKNESS OF CONCRETE UNDER AND AT THE SIDES OF 1-IN. ROUND DEFORMED BAR ON RESISTANCE DEVELOPED

Deformed bar (type No. 4, 1 in. diameter) was embedded horizontally in prisms of cross section shown by the three views at top of figure. All values for 1½ and 2-in. thickness of concrete under bar, are averages for 4 specimens—one each for 2 and 6-in. slump concrete *with* 0-No. 100 material in the sand and one each for 2 and 6-in. slump concrete *without* 0-No. 100 material. All values for 1-in. thickness of concrete under bar are the average of 2 specimens—one each of 2 and 6-in. slump *with* 0-No. 100 material in the sand. Cement content of concrete, 4.3, 5.0 and 6.8 sacks per cu. yd. each with 2 and 6-in. slump. For detailed information on proportions, W/C ratios, sand and aggregate grading see Appendix. All tests at age of 7 days on moist cured specimens, tested damp. Slips indicated are at free end of bar.

The tests with the 2-in. coverage were consistent in showing a greater increase in stress as the slip increased than was shown by the specimens with 1½ or 1-in. coverage. This difference became more marked as the length of embedment and richness of mix were increased. For the richest mix and 22-in. embedment, the specimens with 1-in. coverage showed no increase in stress with increased slip. These differences in behavior and the fact that at low slips the specimen with 2-in. coverage showed less stress than the others, seem to be explained by the opposing effects of two factors. One of these is the amount of concrete under and at the sides of the bar which should increase the resistance as the depth of concrete coverage increased. The other is the settlement of the concrete under the bar during placing which weakens the bond on that side. This factor increases as the depth under the bar increases.

The maximum stress at failure, which in all cases occurred by split-

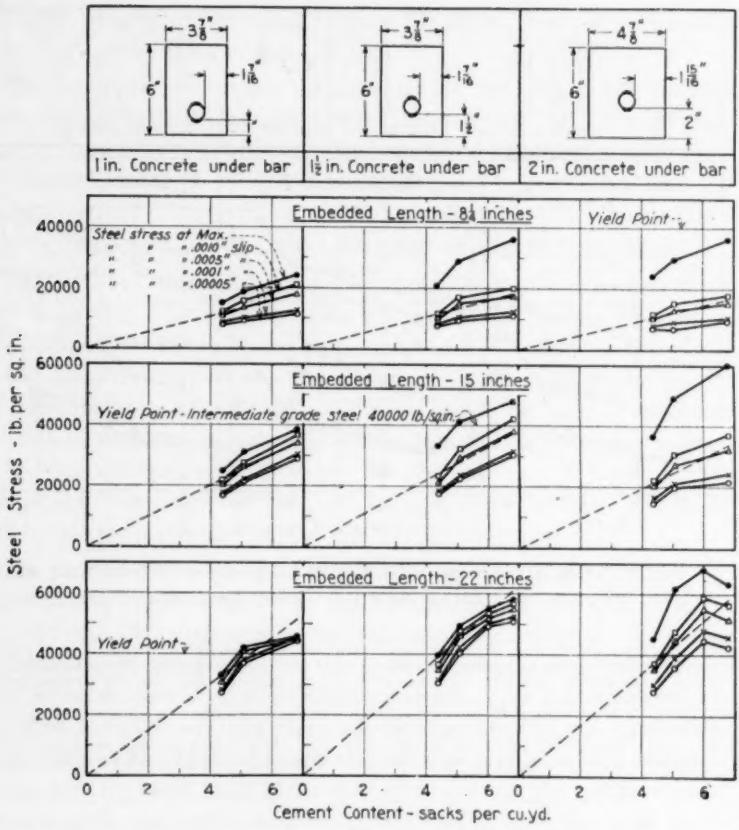

1 in. Concrete under bar
1½ in. Concrete under bar
2 in. Concrete under bar
Embedded Length - 8¼ inches
Embedded Length - 15 inches
Embedded Length - 22 inches
Steel stress at Max.
Yield Point
Yield Point - Intermediate grade steel 40000 lb/sq.in.
Steel Stress - lb. per sq. in.
Cement Content - sacks per cu.yd.

FIG. 12—EFFECT OF CEMENT CONTENT ON RESISTANCE DEVELOPED BY 1-IN. ROUND DEFORMED BAR WITH DIFFERENT COVERINGS OF CONCRETE
See notes, Fig. 11, for details.

ting of the concrete, was consistent in showing higher values for the greater depth of coverage.

Effect of Cement Content on Resistance Developed by 1-in. Round Deformed Bar with Different Coverings of Concrete. The data plotted in the diagrams of Fig. 12 are based on the same tests as in Fig. 11 but are replotted here to bring out the influence of cement content. Included also in the two lower right diagrams are values for tests with concrete with 6.0 sacks of cement per cu. yd.

In each diagram the light dash line is drawn from the origin (representing zero cement content and zero steel stress) to represent the average trend of the steel stresses developed at slip of .0005 in. Similar

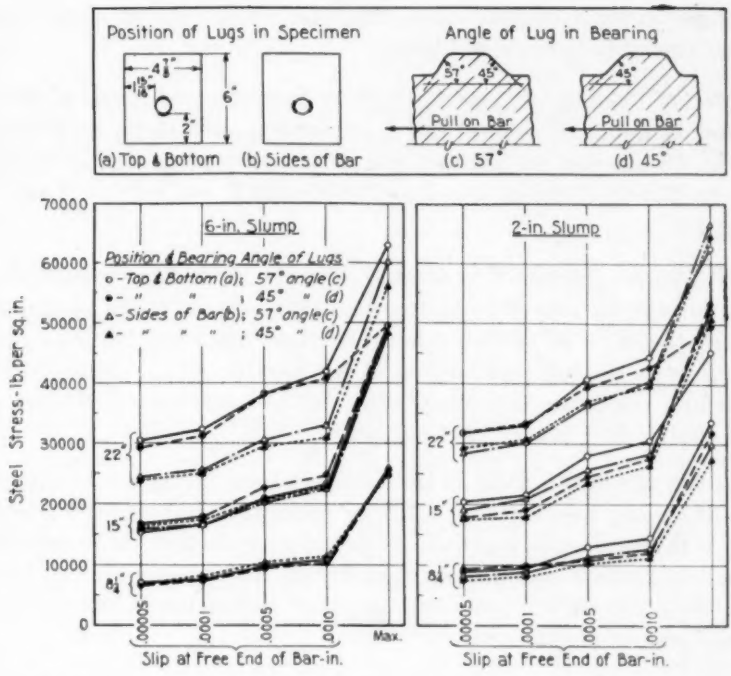

Position of Lugs in Specimen
Angle of Lug in Bearing
(a) Top & Bottom
(b) Sides of Bar
(c) 57°
(d) 45°
Pull on Bar
6-in. Slump
2-in. Slump
Position & Bearing Angle of Lugs
o - Top & Bottom (a); 57° angle (c)
• - " " ; 45° " (d)
Δ - Sides of Bar (b); 57° angle (c)
▲ - " " " ; 45° " (d)
Steel Stress - lb. per sq. in.
Slip at Free End of Bar-in.
Max.

FIG. 13—EFFECT OF POSITION OF LUG (AS CAST) IN SPECIMEN AND ANGLE OF BEARING SURFACE OF LUG

Deformed bar (type No. 4, 1 in. diameter) embedded horizontally in prisms with transverse lugs in the positions shown in (a) and (b) and with the 57° and 45° angle of the lug brought into bearing as shown by (c) and (d). Concrete: 5.0 sacks cement per cu. yd.; 2 and 6-in. slump (W/C = 0.59 and 0.64 by weight). Specimens moist cured 7 days and tested damp. Slips indicated are at free end of bar. Data shown are based on one specimen for each casting position.

lines could be drawn for stresses at other slips. It will be seen that the points representing the observed stresses would follow these lines fairly well from the leanest to the richest concrete shown, particularly for the bars embedded 8¼ and 15 in. For the 22-in. embedments the agreement is not so good although for a range of cement content of from 4.3 to 6.0 sacks per cu. yd. the resistance developed is closely proportional to cement content.

The general trend for increase in steel stress in proportion to increase in cement content was indicated by all three depths of coverage.

The stresses at maximum indicate that the resistance to splitting is approximately proportional to cement content.

In this connection it may be well to point out that most of the test results reported in the various figures of this paper are based on test specimens containing 5.0 sacks of cement per cu. yd. With specimens

of higher cement content than this, the resistances or steel stresses would probably have been higher.

Effect of Position of Lug (As Cast) in Specimen and Angle of Bearing Surface of Lug. In general the tests for 8¼ and 15-in. embedments in Fig. 13 showed no large effects, of either the position or angle of lug, on the steel stress developed either with concrete of 6 or 2-in. slump.

With 22-in. embedments somewhat lower steel stresses were developed for the various slips at free end when the bar was placed with lugs at the sides of the bar (detail b) than when the bar was placed with lugs at the top and bottom of the bar (detail a). However, the maximum resistance developed with lugs at sides compared favorably with that of bars placed in the other position.

On the whole the data do not indicate that variations in lug position or in lug bearing angles tested constitute important factors in bars with transverse lugs. It is practically certain, however, from other tests which are not reported herein that angles greater or less than those shown will produce important differences in the resistance characteristics of the bars.

Influence of Horizontal and Vertical Casting Position and of Rodding Disturbance on Resistance Developed by 1-in. Round Deformed Bar with Transverse Lugs. Each diagram of Fig. 14 shows a set of 3 curves through plotted points, one curve for each of the three casting positions illustrated by details (a), (b) and (c) in the upper left diagram.

It will be seen from the load-slip curves (based on slip at free end) that the stresses required to produce a given end slip are in the main markedly different for each casting position. The highest steel stress for a given slip was developed in every case for position (c) and the lowest steel stress for position (a). In position (c) the concrete settled in a direction *opposite* to pull on bar and *against the lugs* whereas in position (a) the concrete settled in the *same direction* as the pull on bar and *away from or under the lugs.* Thus in position (c) the concrete was in intimate contact with the lugs providing for more effective interaction between steel and concrete almost from the first application of pull on the bar. With position (a) the bar had to slip appreciably before the lug was brought into bearing with the concrete. Before bearing became effective the resistance developed depended mainly on that offered by the surface of the bar.

The specimens which were cast with the bars horizontal, position

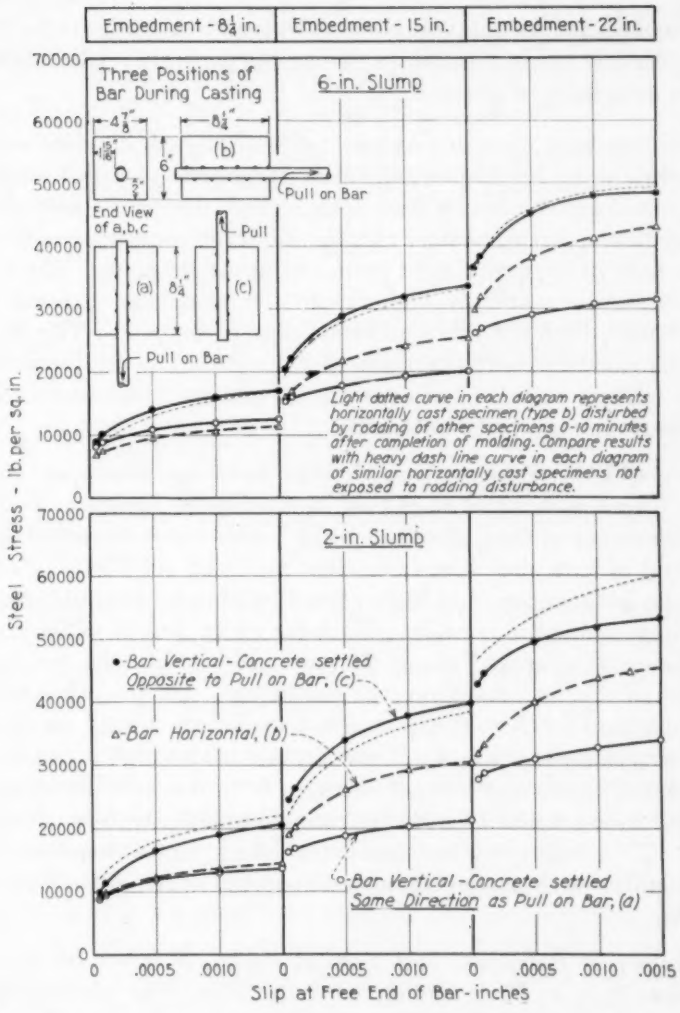

Embedment - 8¼ in.
Embedment - 15 in.
Embedment - 22 in.
Three Positions of Bar During Casting
6-in. Slump
(b)
Pull on Bar
End View of a,b,c
Pull
(a)
(c)
Pull on Bar
Light dotted curve in each diagram represents horizontally cast specimen (type b) disturbed by rodding of other specimens 0-10 minutes after completion of molding. Compare results with heavy dash line curve in each diagram of similar horizontally cast specimens not exposed to rodding disturbance.
2-in. Slump
•-Bar Vertical - Concrete settled Opposite to Pull on Bar, (c)
▵-Bar Horizontal, (b)
○-Bar Vertical - Concrete settled Same Direction as Pull on Bar, (a)
Steel Stress - lb. per sq. in.
Slip at Free End of Bar - inches

FIG. 14—INFLUENCE OF HORIZONTAL AND VERTICAL CASTING POSITION AND OF RODDING DISTURBANCE ON RESISTANCE DEVELOPED BY 1-IN. ROUND DEFORMED BAR WITH TRANSVERSE LUGS

Deformed bar (Type No. 4). Bars were embedded 8¼, 15 and 22 in. in prisms of the same cross-section, 4⅞-in. wide and 6 in. high as shown in end view. Concrete: 5.0 sacks cement per cu. yd.; 2 and 6-in. slump (W/C = 0.59 and 0.64 by weight). Specimens moist cured 7 days; tested damp.

(b), (heavy dash lines) developed resistances at a given slip intermediate between those developed by bars in the vertical positions (a) and (c). Although the concrete settles under the horizontal bars the settlement is not as unfavorable with respect to bearing under the lugs as for vertical bar in position (a) where the concrete settles under each lug on both sides of the bar.

The very light dash-line curves included in each diagram represent load-slip curves for horizontally cast specimens identical with those represented by the heavy dash lines except that the specimens were subject to the disturbance of rodding adjacent specimens as described in the note in the lower right corner of upper diagram. The increase in resistance is markedly higher with the specimens exposed to this disturbance, both for the 2 and 6-in. slump concrete. This factor of disturbance after completion must be recognized as an important one in the making of test specimens and should be eliminated entirely if reliable results are to be obtained.

Effect of Water Leakage from Mold on Resistance Developed with 1-in. Plain Round Hot Rolled Bars Cast in Horizontal Position. In Fig. 15 the resistances of bars placed in leaky molds are represented by dash lines and of bars placed in water-tight molds by solid lines. It is seen from the left diagram that higher bond resistances were obtained with the leaky molds for the 6-in. slump for every length of embedment. This increase averaged about 24 per cent whether the resistance at various slips or at maximum was considered. On the other hand the right diagram for 2-in. slump concrete shows practically no difference in resistance for leaky and water-tight molds although it can be definitely stated that water leakage occurred from the 2-in. slump concrete in lower amount and at a slower rate than with the 6-in. slump concrete. This improved resistance for leaky molds requires further investigation with particular attention to the depth of concrete under the bar.

Influence of Horizontal and Vertical Casting Position on Resistance Developed by 1-in. Plain Round Hot Rolled Bars. The tests represented in Fig. 16 show the effect of position of bar during casting of the specimen on the stress at different amounts of end slip. These tests which are for plain bars are similar to those for deformed bars in Fig. 14. The data, however, are presented in a different manner. They also include tests for bars embedded for 30 in. (30 diameters).

The two upper diagrams show the relation between the resistance of bars held in position "c" during casting (ordinate) and that of bars

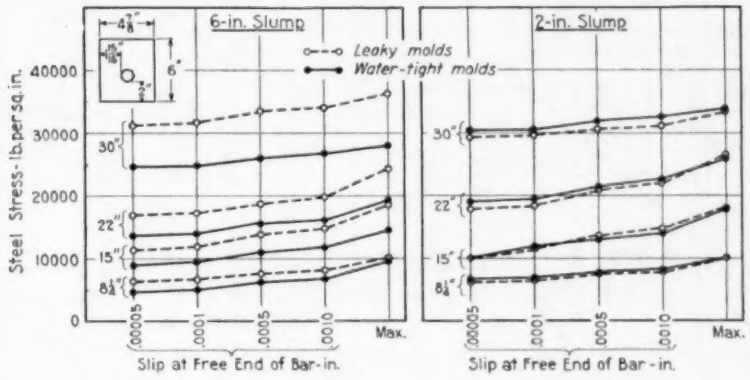


FIG. 15—EFFECT OF WATER LEAKAGE FROM MOLD ON RESISTANCE DEVELOPED WITH 1-IN. PLAIN ROUND HOT ROLLED BARS CAST IN HORIZONTAL POSITION

Bars (Type No. 2) were embedded horizontally in 8¼, 15, 22 and 30-in. prisms 4⅞-in. wide and 6 in. high as shown in the upper corner of left diagram. Concrete: 5 sacks cement per cu. yd.; slump 2 and 6 in. (W/C ratios of 0.59 and 0.64 by weight). Specimens moist cured 7 days; tested damp. Data shown based on one specimen for each condition of test. Slips indicated are at free end of bar. The water-tight molds were sealed with a 50-50 mixture of hot paraffin and rosin. After the concrete had been placed, rodded and struck off *the "leaky" molds* were produced by scraping away the sealing compound from the joints between the side, end, and bottom plates. The cap screws holding the base plate to the end plates were also loosened.

held in position "a" (abscissa). In the two lower left diagrams a similar relation for positions "c" (ordinate) and "b" (abscissa) are shown. In these diagrams the plotted points can be compared with a light dash line representing equality of values.

The upper diagrams for 6 and 2-in. slump show that there was no substantial difference between the bond resistance developed by the plain bars cast vertically whether the concrete settled either in the same direction as the pull on bar, or in the opposite direction.

The two lower left diagrams show much higher resistances for the vertical bars than for those held horizontally during casting, particularly for the shorter embedments. Ratios of the resistance of the horizontal and vertical bars are tabulated in the lower corner of each diagram for each embedded length. This tabulation for 6-in. slump concrete indicates that horizontal bars embedded for 8¼ in. only develop 48 per cent of the resistance of vertical bars but that for 15 22 and 30 in. embedments the resistance rises to 52, 57 and 72 per cent respectively. For the concrete of 2-in. slump a similar improvement is shown in the ratio between the resistance of horizontal and vertical bars as the length of embedment increases. The ratio, however, is definitely higher for all embedments.

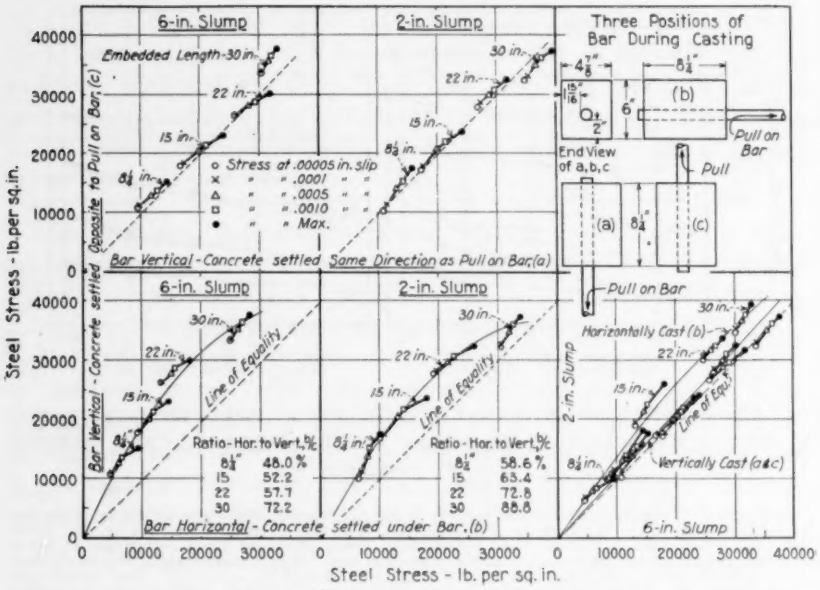


FIG. 16—INFLUENCE OF HORIZONTAL AND VERTICAL CASTING POSITION ON RESISTANCE DEVELOPED BY 1-IN. PLAIN ROUND HOT ROLLED BARS

Plain round bars (Type No. 2) embedded for 8¼, 15, 22 and 30 in. in prisms of the same cross-section, 4⅞-in. wide, and 6 in. high, as shown in end view in this figure. Concrete: 5.0 sacks cement per cu. yd.; 2 and 6-in. slump (W/C = 0.59 and 0.64 by weight). Specimens moist-cured 7 days; tested damp. All molds were water-tight with no leakage from specimens. Data shown based on one specimen for each condition of test. Slips indicated are for the free end of the bar.

In the lower right diagram the resistance values for 2-in. slump concrete are plotted as ordinates and those for 6-in. slump concrete as abscissas to show the effect of slump with different casting positions. The plotted points show no substantial effect of slump for vertical bars either in position (a) or (c) since the points fall practically on the line of equality. The plotted values for horizontal bars (position b) on the other hand show that higher resistances are generally obtained with the 2-in. slump concrete than with the 6-in., as indicated by the location of the light full line (average) curve with reference to the line of equality.

Comparative Results Obtained in Bond Tests at Different Periods—1912-1937. The upper diagram of Fig. 17 shows the maximum bond resistance developed in pull-out tests plotted against compressive strength of the concrete. For a given compressive strength, it is seen that tests with 1937 cement and 1937 bars show about half the bond

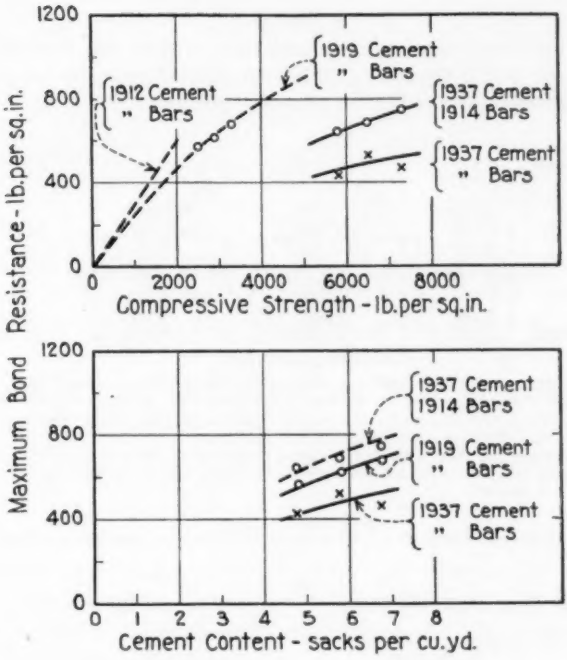


FIG. 17—COMPARATIVE RESULTS OBTAINED IN BOND TESTS AT DIFFERENT PERIODS—1912-1937

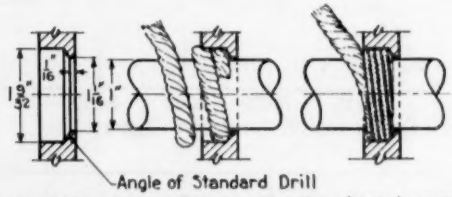
Bars 1-in. plain round hot rolled. Specimens: 8 by 8-in. cylinders cast with bars in vertical position for settlement of concrete in same direction as pull on the bar. Graph for 1912 cement and bars is from Bulletin 71, University of Illinois Engineering Experiment Station. Graph for 1919 cement and bars plotted from data in Bulletin 17, Structural Materials Research Laboratory, Lewis Institute. 1914 bars used with 1937 cement were furnished by Prof. F. E. Richart from old stock still available at the University of Illinois.

resistance found in the tests made in 1919 using cement and bars of that time. Intermediate between these are the results of tests made with 1937 cement and 1914 bars.

The lower diagram shows maximum bond for the same groups of tests plotted against cement content in sacks per cu. yd. On this basis it will be seen that the differences in bond between the 1919 and 1937 tests are relatively much less than when compared on the basis of compressive strength. It will be seen also that when the 1914 bars were used with 1937 cement, the bond resistance was greater at a given cement content than in either of the other two groups of tests.

APPENDIX

The following notes supplement information given in the introduction on the type of concrete used. They give further details relative to the proportion of the mix and the gradation of the Elgin sand and gravel aggregate.



Clearance between bar and recessed portion of hole is caulked with ¼-in. hemp rope rammed tight with caulking tool. A 1-in. loose end of rope provides a grip for pulling wedged portions from recess when stripping mold. When considerable hydraulic pressure is developed or the mold is subject to vibration a specially water-tight joint may be obtained by applying sealing compound to each turn of rope as it is caulked.

FIG. 1A—DETAIL OF SEALING MOLD AT POINT WHERE BAR PASSES THROUGH THE ENDS

Cement Content Sacks per Cu. Yd.	Mix by Weight	Material Proportions by Weight			% Sand 0-No. 4	W/C Ratio by Weight		Compressive Strength, lb. per sq. in.			
								7 Days		28 Days	
		Cement	Sand	Coarse		2-in. Slump	6-in. Slump	2-in. Slump	6-in. Slump	2-in. Slump	6-in. Slump
4.3	1–8.33	1	3.58	4.75	43.0	.72	.76	——	——	——	——
5.0	1–7.0	1	2.80	4.20	40.0	.59	.64	4300	3600	6000	5300
6.0	1–5.8	1	2.24	3.56	38.5	.50	.55	——	——	——	——
6.8	1–5.0	1	1.85	3.15	37.0	.45	.49	5700	5500	7100	6800

The sand was graded 0-No. 4 sieve as follows: 40 per cent by weight of 0-No. 28; 25 per cent No. 28-14; 17 per cent No. 14-8; 18 per cent No. 8-4. The grading of the 0-No. 28 material was maintained as follows: 15.1 per cent by weight of 0-No. 100; 12 per cent No. 100-48; 72.9 per cent No. 48-28. The grading of the 0-No. 100 material was maintained to provide 22 per cent by weight of 0-No. 200 and 78 per cent of No. 200-100. In some series of tests (as indicated in the text) the 0-No. 100 sand was purposely omitted from the 0-No. 28 material to obtain a substantial variation in fines. The usual percentage of 0-No. 100 material based on total aggregate in the mix was as follows: 2.60, 2.42, 2 32, and 2.24 per cent respectively for mixes designated as 4.3, 5.0, 6.0 and 6.8 sacks per cu. yd.

With 1½- and 2-in. clearances between bar and mold the coarse aggregate was graded from No. 4 to 1 in. as follows: 25 per cent by weight of No. 4-⅜-in.; 50 per cent ⅜-¾ and 25 per cent ¾-1 -in. With clearances of 1-in. the coarse aggregate was graded from No. 4 to ¾-in. as follows: 25 per cent by weight of No. 4-⅜-in. and 75 per cent ⅜-¾-in.

Discussion, to close in the September Supplement, should reach the A. C. I. Secretary by July 20.

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(Supplement)

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7400 SECOND BOULEVARD, DETROIT, MICHIGAN *SEPTEMBER 1939*

Discussion of a paper by Carl A. Menzel:

Some Factors Influencing Results of Pull-Out Bond Tests*

BY A. E. LINDAU, F. R. MCMILLAN, F. E. RICHART, M. A. SWAYZE, DUFF A. ABRAMS, M. O. WITHEY, H. J. GILKEY AND CARL A. MENZEL (AT THE CONVENTION); CHESLEY J. POSEY AND AUTHOR'S CLOSURE

EDITOR'S NOTE

There was unfortunate transposition of illustrative material in publishing Mr. Menzel's paper in the June JOURNAL: Fig. 3 appears over the caption for Fig. 4 and vice versa—p. 524. On page 526 a similar mistake was made with Fig. 6 and 7.

CONVENTION DISCUSSION

*A. E. Lindau**—I believe that this is a very important paper; it affects our whole method of design, or at least it affects the degree to which we can increase or decrease our stresses, and it affects our confidence in our design. We more or less quit thinking about bond a good many years ago, because we thought the story was pretty well told. Yet we were still faced with the problem that Mr. Menzel has, to some degree, solved. This country took a decidedly different view from the old practice where a plain bar was considered sufficient in bond resistance. I think it was very fortunate that the early users of reinforced concrete had the idea that bond was sufficiently important to make an effort to improve it over that which we could get with a plain bar. When we evaluate the results of this type of research, we naturally tie back to our experience, and then we try to see whether our experience will, to some degree at least, check with the new research. It is a very difficult thing to check up bond from experience.

*JOURNAL, Amer. Concrete Institute, June 1939; *Proceedings* Vol. 35, p. 517.
*Chicago.

Bond failures probably would be represented by excessive or dangerous cracks. We have cracks in concrete structures that are frequently difficult to explain, and we cannot say that they are due to a bond failure. We have had tests with failure in diagonal tension; we ascribe it to the bond, and if there is one scale in our armor of reinforced concrete design that I fear is weaker than some of the others, I would say it is the matter of diagonal tension which began with dependence on the bond. I believe that we may have from this paper and perhaps others, the information that we sought in 1920 or 1924. to define deformed bars more closely. It was necessary to eliminate variables in our bond tests in order to get at that. It was also necessary to find a basis on which a specification or a method of design or recommended design could be made. I believe that we have that now, and I believe it is sufficiently important, if the manufacturers and others interested in reinforcement material, take this thing seriously enough, to do something about it. Unfortunately the general attitude has been that any kind of a deformation or lug classes it as a deformed bar, and in the absence of definite information we could take issue with that view. Now I believe we can do so.

Chairman McMillan—I should say, for Mr. Menzel that this represents only a small part of the work already done. A very large program is under way and there will be other contributions on the subject of bond in the future.

*F. E. Richart**—The information shown on Mr. Menzel's chart is usually expressed in terms of steel stress rather than bond stress—I should like to ask him whether he has made a study of the effect of imbedment on the steel stress. That is important, because if we try to set up any kind of a standard test for deformed bars, we certainly will need to determine several things, including the amount of slip permitted and the amount of imbedment in terms of the diameter of the bar.

C. A. Menzel—These tests have shown that we cannot establish the resistance characteristics of any bar on the basis of only one length of embedment. To be fair we should have three, possibly four, lengths of embedment in any series of pull-out tests, and in any case the length of the embedment should be such that we at least develop a steel stress equal to or greater than the yield point in the steel. Unless you do that, you really do not know very much about the characteristics of the bar. As regards the method of expressing the results on the basis of bond per unit of surface area, that can be

*Research Professor of Engineering Materials, University of Illinois, Urbana.

easily done from the data presented. In general, the resistance characteristic of the bar is disclosed by the shape of the curve; if the curve for steel stress is concave upwards, then the curve for bond resistance in pounds per square inch of surface is also concave upwards; if the curve is concave downwards, then the longer the embedment, the lower will be the average resistance of the bar in pounds per square inch of bar surface.

*M. A. Swayze**—I should like to ask Mr. Menzel if there has been any attempt at the correlation of the bond strength with the yield point of the steel used? I saw in Germany this past summer some concrete joists reinforced in the bottom by 23 piano wires which had been pre-stressed. The failure of those bars on test was very unusual because there was absolutely no pull-out with the exception of about, I would say, six wires in some 20 beams which I looked at. The break of the wire was absolutely flush with the break in the concrete, and it occurred to me that pre-stressing and then the release of that stress when the wires were cut, would cause a very slight increase in the cross section of those wires, which probably increased the bond, simply due to the enlargement of the cross section, and if that should be the case, the yield point of the steel used would have some effect on the bond.

Mr. Menzel—It was very clear as the tests progressed that there was a very great difference in the yield point of steel that was supposed to have a constant yield point. That is what makes these tests so difficult; you never know when you are dealing with the same material, and commercially deformed bars, especially, are so variable that it is very difficult to get uniform results; the character of the surface between lugs varies, the shape and size of the lug varies; the yield point of the steel varies, and there are so many other factors that come into play.

Duff A. Abrams†—Mr. Menzel has kindly referred to tests I carried out 30 years ago. My interest in bond in recent years has not been very great, but I do wish to compliment Mr. Menzel on this very extensive program. I wonder if he has made any efforts towards carrying out determinations of bond more nearly as it exists in the case of a beam? In other words, after all, the pull-out test is only a very indirect symptom, I think, of what happens in beams. In a beam slipping is more or less of an instantaneous nature. Concrete stresses are entirely different, in general, from stresses sustained in a pull-out specimen, so that the whole picture of the bond resistance

*Lone Star Cement Corp., Hudson, N. Y.
†Consulting Engineer, New York.

with reference to stress is quite different. In other words, as the concrete is in tension at the same time the steel is in tension, the first slip is progressive from point to point, so that under those conditions the stress which would exist in a very short embedment would be a very great deal more to the point than that developed by a very long embedment.

Mr. Menzel—It should be pointed out perhaps that in these tests of slips at the bottom of the specimen or at the loaded end of the bar, slips occurred with the smallest load that we could apply to the bar. This slip is progressive almost from the very beginning; slip occurs at the loaded end long before it becomes evident at the free end. That is precisely in accord with what your tests on beams showed so thoroughly in University of Illinois Bulletin 71. Now, regarding the validity of the pull-out test, I think that has been argued pro and con for about 30 years. I remember that you were one of the defenders of the pull-out test in at least one concrete session, in a discussion with Professor Slater. Admittedly, the pull-out test is not a test that can be extended directly to the results you would get in beams.

There are so many different combinations in the assembly between the concrete and the steel, that it looks to me that for the present at least while there are still so very many factors involved, that we should continue with the pull-out tests. It is well to remember the following: Bond resistance really is a resistance developed between the surface of the steel and the concrete and this resistance is what we rely upon to establish the necessary intimate inter-action between the two materials and without which we cannot have the steel effective at all. The pull-out test does give a pretty reliable index of the relative resistance developed by bars of various designs, of concrete of different mixes and all the other factors that come into the picture. If we are going to rely on developing the influence of similar factors with beam specimens, it would be many, many years, I am afraid, before we would really have the information which we can get in at least one-tenth of the time with the pull-out test, and at a much lower cost. With the information that is being developed from the pull-out tests it is hoped that ultimately the desired information can be obtained from a relatively few well-chosen beam-type specimens.

*M. O. Withey**—I am glad that Mr. Abrams brought up the point regarding tests with beams. I appreciate thoroughly the stand just taken by Mr. Menzel, that the pull-out test saves time and gives a great deal of information at a minimum expense, that would take a

*University of Wisconsin, Madison.

very long time to get with the beam test. After all, when all is said and done, you apply the results of tests in designing slabs, beams, girders, columns and other members as well, so I think before finishing the program of bond experiments, it would be well to have some beam results and information on a subject which perhaps is more vital. It seems to me that it would be highly desirable to ascertain what bond resistance is under continued loading; that is, to have beams which could be maintained loaded for a time with a simple arrangement, to see if the bond deteriorates.

Chairman McMillan—I will say that in this study we had hoped, before this, to be at work on beams, but had we hurried into this phase of the subject part of the work would have been lost completely. It was only through the early work on the pull-out specimens that we developed some of the points Mr. Menzel has brought out. We have had some pull-out tests under continued load, but only a few, not sufficient to justify reporting at this time. We will certainly have some beam tests in our future work.

Mr. Abrams—As to pull-out tests; I still take the same stand, but I reserve the right to interpret the tests after I get them. I think the point Mr. Menzel had in mind probably goes back to the fact that in the early days many people made pull-out tests on very small bars, probably ⅜ in. or ½ in., and the consequence was they over-stressed their steel long before they developed what they considered bond failure. So I think I stated at that time that as long as you used a bar, say an inch in diameter, or larger, embedded 8 in. in an 8-in. diameter specimen, you were getting a pretty fair measure of bond resistance as far as the type of bar and concrete and other factors are concerned.

*H. J. Gilkey**—I cannot resist speaking on the pull-out tests versus beams. Numerical differences between the bond resistances developed are apparently due, largely if not entirely, to the differences in the normal casting positions for these two types of specimen. The vertically cast pull-out specimen does invariably develop a higher bond resistance than does the horizontally cast beam with the settling away of the concrete and the water gain beneath the steel; even if there is not actual honeycombing there. We have within the past few months reported test results before this society which seem to indicate that, for specimens similarly oriented at casting, beams and pull-out specimens may be expected to give identical values for the bond resistance developed. Moreover, we have shown that tests by others indicate excellent correlation between the values secured from vertically cast

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pull-out specimens and horizontally cast beams. Thus, while in these latter comparisons the pull-out values are higher, the nature of the action is exactly the same; the relationship on a diagram being, within the variability of test conditions, a straight line. Such comparisons seem to have demonstrated beyond reasonable question that the pull-out type of specimen is an entirely proper and satisfactory one for use as a relative measure of bond resistance under beam conditions. Certainly the pull-out specimen is much the simpler to fabricate, handle, and test.

CHESLEY J. POSEY*

The writer believes that comparisons of bond resistance by pull-out tests are most significant when based upon slip measurements taken at the loaded end of the embedment.† They are seldom made on this basis, however, because of the difficulty of measuring the slip there, and the comparative ease of measuring the slip at the free end. Mr. Menzel describes an ingenious yet straightforward method for measuring the slip at the loaded end of the embedment. In Fig. 9, he illustrates the best method of comparing the effect of different lengths of embedment; a plot of the load-slip curve based on slip at the loaded end. Similarly, in Fig. 10, he shows load-slip curves which afford a cogent comparison of the value of different types of surface roughness.

To facilitate comparison of the value of different types of surface roughness, the variable effect of length of embedment may be eliminated. Referring to Fig. 9, it is seen that the load-slip curves for the 15 and 22-in. lengths of embedment (of the same type of bar) are practically identical up to slips at the loaded end of 0.009 in., in the concrete having 2-in. slump. If a slip of 0.01 in. (at the loaded end) is accepted as critical, then, an embedded length of 22 ins. should be sufficient to eliminate the variable effect of embedded length, in comparisons of the bar tested with other bars having surfaces of greater effective roughness. For smoother bars or a weaker concrete, a longer embedded length would be necessary.

In the type of specimen used by Mr. Menzel, the bond between the concrete and the steel is likely to be increased by the transverse compressive stress in the concrete due to the restraint at the bottom end of the specimen. This effect, which was probably not important in Mr. Menzel's tests, may be eliminated by using the type of specimen described by T. D. Mylrea.‡

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*Assistant Professor, Hydraulics and Structural Engineering, State University of Iowa, Iowa City, Iowa.

†Dr. Fritz Emperger, Beton und Eisen, June 20, 1935, pp. 197-200.

‡T. D. Mylrea "The Carrying Capacity of Semicircular Hooks" *Proceedings* American Concrete Institute, Vol. 24, 1928, p. 240.

Mr. Menzel's tests show that roughness of the bar surface may increase the bond resistance as much as 50 per cent. The writer has shown that an artificial roughness of special type may increase the bond resistance of plain round bars far above that of the best available commercial bar. Mr. Menzel's bar 4, which has the best bond resistance of the round bars for which he reports slip at the loaded end, developed an average bond stress of about 400 lbs. p.s.i. at a slip of 0.01 in. at the loaded end. The 1-in. round bar was embedded 22 ins. In the writer's tests* four $\frac{5}{8}$ in. round bars of the same type embedded 13.75 ins. ($L/D = 22$) developed an average bond stress of about 420 lbs. p.s.i. at the same slip. The average strength of the control cylinders was 2320 lbs. p.s.i. The values 400 and 420 are in good agreement, considering that the results are not strictly comparable because of the difference in bar size and the concrete in which the bars were cast. The writer's plain round bars, roughened with a cold chisel, and tested in the 2320-lb. p.s.i. concrete, developed an average bond stress of about 630 lbs. p.s.i. at a slip of 0.01 in. at the loaded end.

The writer is interested in Mr. Menzel's statement that improvement in bond resistance due to the use of the stiffer mix can only be expected when the concrete is placed by hand rodding, or by "some types of vibration." The writer has found indications that vibration may result in a pool of wet concrete immediately surrounding the bar, resulting in abnormally low values for bond. He has never had an opportunity to investigate the phenomenon further, and wonders if Mr. Menzel's qualification of his statement is based upon a similar observation.

Mr. Menzel's data on the influence of casting position, thickness of concrete, and water leakage, and his comparisons with the results of previous investigations are especially interesting. His data on the effect of cement content provide additional evidence in support of the practice of correcting for the effect of small variations in the quality of the concrete used in test specimens by increasing or reducing observed values in proportion to the strength of control cylinders.

Mr. Menzel presents no data on the variability of results from supposedly identical specimens. The variability found in investigations of the compressive strength of concrete is sufficiently large to be of importance. The relative variability of bond strengths is probably even greater.

AUTHOR'S CLOSURE

Mr. Posey refers to the writer's statement that with stiff mixes the improvement in bond resistance can only be expected when the

*C. J. Posey "Tests of Anchorages for Reinforcing Bars" University of Iowa Studies in Engineering, Bulletin 3, 1933.

concrete is placed by hand rodding or "by some types of vibration." In answer to his question the author would state that the qualification as to "some types of vibration" was deliberate. It was found that much depended on the source and duration of the vibration and the manner in which it was applied. Both increases and decreases in bond resistance were shown for different methods compared to careful hand rodding.

Mr. Posey raises an important point in his comment on the variability in results between supposedly identical specimens. As in the case of other types of concrete tests, the tests for bond strength are subject to the numerous factors which influence the properties of concrete as well as to those factors which are peculiar to the making and testing of specimens with embedded bars. It was found early in the study that consistent and dependable results could only be obtained when all the operations of making, placing, curing, and testing were carried out in accordance with a rigid procedure. With the procedure adopted results were obtained which were comparable in variability with those usually obtained in concrete testing under well controlled conditions.

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Paint for Durability on Concrete Surfaces*

By Paul O. Blackmore†

SYNOPSIS

Fourteen hundred concrete blocks were made up according to specifications, both as to composition and cure. After ageing two weeks, blocks were painted, using nine different types of primers and nine topcoats in all possible combinations with each other. All primers were also tested as topcoats over themselves and all topcoats were tested as primers. All of the combinations were pigmented using red oxide, toluidine red, chrome green, ultramarine blue, iron blue, chrome yellow, chromium oxide, mapico yellow, white.

Total exposure to weather, three years to date, and 124 combinations still on the roof at Cincinnati. A smoky industrial atmosphere prevails, adjacent to a chemical plant. Results are discussed both in regard to vehicle and pigment formulation, and inferences drawn. All blocks were painted in duplicate, representing, therefore, 700 different combinations.

INTRODUCTION

Whereas it is the job of you of the American Concrete Institute to create structures, it is mine to help beautify and protect them. In this we have the cooperation of one great industry with another for the development of information of common value to both. My little contribution is but a fragment, but it enlists me in your army of knowledge seekers.

There are a number of reasons for painting concrete. Beautification, protection against the ravages of sun, wind, and rain, rejuvenation after the collection of dirt, prevention of water seepage, are only a few of them.

*Presented 35th Annual Convention American Concrete Institute, New York, March 1-3, 1939.

†Chief Chemist Ault & Wiborg Corp., Cincinnati, Ohio.

Suffice to say that it often becomes necessary to paint concrete structures and that such procedure is frequently fraught with headaches. It has been the practice to do the painting with a prayer, cross your fingers and await the worst, and sometimes the worst was not long in coming—in the form of severe blistering and peeling on the side of a building, and we said that free lime in the concrete had "burned" the paint film causing it to fail, and we offered our apologies and hoped they'd let it go at that.

It was an unsatisfactory condition, and because of it a program of research was agreed upon between the Portland Cement Association and the Ault and Wiborg Corp., the purpose of which was to develop a system for painting concrete which could be applied to the concrete in a comparatively short time after it had hardened, without surface treatment of any sort and one that would resist failure by cracking, blistering, peeling and lime burning for a length of time considered reasonable for exterior paints.—three to five years.

SPECIFICATIONS FOR BLOCKS

It was accordingly agreed that the Portland Cement Association should have prepared according to its specifications and subject to its approval, 1400 concrete blocks, 8 x 8 x 1½-in. One face was cast against a rough form board, the other steel-troweled. Six and one-half gallon concrete was used with a five to six inch slump, and having the approximate proportions of 1-2¾-2¾ by weight; maximum size of the coarse aggregate ½ in. The steel-troweled surface had a delayed finish to remove any laitance. The blocks were moist cured seven days followed by seven days of air-drying, and delivered for painting at the rate of 48 per day. This procedure was recommended to keep the moisture content constant within reasonable limits.

The job of making the blocks was let to a Cincinnati contractor who prepared approval samples for the Portland Cement Association before starting production on the 1400 blocks.

SPECIFICATIONS FOR PAINTING

All paint materials were applied immediately upon delivery of the blocks at the end of the seven day dry cure. It was realized that painting the concrete after this short ageing period would tax the paint durability severely, but it was reasoned that inasmuch as many contractors, in actual practice, would not allow more than one week before painting, it would be well to test the paint under these conditions.

All blocks were painted in duplicate. The paint was applied by brushing and each block received two coats. It most cases priming paints were used of a composition considerably different from that of

the topcoat. In some cases, however, the top-coat was used directly on the concrete, the block receiving a second coat of the same material. In all cases, one week elapsed between application of the primer and the topcoat, and another week for the drying of the topcoat before the blocks were put out for exposure. While the latter air dry period is not possible in actual practice, it must be remembered that the object of the investigation was to evaluate a large and varied number of paint materials against each other under conditions as nearly uniform as possible. The drying, probably the most important condition of all, had, therefore, to be controlled if the results were to be comparable. The painting of the blocks took place between Oct., 1935 and mid-January, 1936, during which period exterior drying conditions would have varied considerably.

The blocks were exposed on the roof of the Ault and Wiborg Corp. building in the heart of Cincinnati and in an industrial section of the city, where smoke and industrial fumes are prevalent. The plant is adjacent to railroad tracks, a chemical plant and a soap manufacturer, all of which contribute to the pollution of the atmosphere. The blocks were exposed at an angle of 45 degrees, facing south, which is standard testing practice.

Cincinnati weather is generally unpredictable but temperatures vary from an average low of 15°F. during winter months to an average high of 90°F. during the summer. However, during the exposure period of these blocks, they went through a winter of abnormally low temperatures, the lowest point reaching —17°, and through an abnormally hot summer reaching a high of 108°. For two weeks the thermometer did not drop below 100. Another winter brought abnormal snow and rainfall, which caused the level of the Ohio river to rise to 80 feet, inundating Cincinnati and the whole of the Ohio valley and setting a rew record for flood crests in that area. It would be hard to find a set of more severe exposure conditions than those prevailing during this test.

PAINT MATERIALS USED

The priming paints were selected with the intention of imparting as much alkali and water resistance as possible to the system without affecting adversely the film flexibility necessary to provide good exterior durability.

It was decided to investigate four of the most outstanding phenol-formaldehyde resins offered by the leading resin manufacturers. These were cooked into varnishes of medium oil length. Three of the varnishes were pigmented in an approved manner for primers for

exterior exposure and to the fourth was added aluminum bronze in the proportions of two pounds to a gallon. Other primer vehicles investigated were:

A varnish prepared from an alkali-resisting hydrocarbon resin;

Two types of rubber derivative resins;

A pigmented lacquer-type primer.

A clear bakelite-lacquer sealer.

An alkyd resin emulsion.

A long oil pigmented primer which had given a good account of itself in previous tests.

For purposes of further identification elsewhere in this paper, we shall assign the following numbers to them:

TABLE 1—PRIMERS

No. 1	Long oil pigmented primer.
No. 2	Phenolic varnish A with aluminum bronze.
No. 3	Phenolic varnish B pigmented.*
No. 4	Hydrocarbon resin varnish pigmented.
No. 5	Phenolic varnish C pigmented.*
No. 6	Phenolic varnish D pigmented.*
No. 7	Rubber derivative resin No. 1.
No. 8	Rubber derivative resin No. 2.
No. 9	Pigmented lacquer primer.
No. 10	Clear bakelite lacquer sealer.
No. 11	Alkyd resin emulsion.

*Phenol-formaldehyde resins cooked with China wood oil and all pigmented in the same manner. The ratio of oil to resin the same in each case and the amount of pigment to binder the same in all three.

For the finishing coats, it was decided to select a linseed oil paint of approved quality and one which had given a good account of itself in previous tests and modify it with an alkyd resin, two more phenol-formaldehyde resin varnishes, and a specially treated China wood oil. These various modifications were pigmented with two different combinations of whites, chrome green, chromium oxide green, Mapico yellow, toluidine red, two types of red oxide, and tints of the white base with iron blue and ultramarine blue. For purposes of further identification these are shown in Table 2:

TABLE 2—FINISHING COATS

A	Straight linseed oil paint.
B	Alkyd resin modification.
C	Phenolic varnish E.
D	Phenolic varnish F.
E	Treated China wood oil.

All of the modifications were pigmented with all of the above dry colors, and all combinations of the two were applied over all primers. In addition, all colors of all finishing coats were applied over themselves, and two of the primers, 1 and 2, were used over themselves, serving as both primer and finishing coat. All combinations of all

formulations in all colors were exposed, representing between 1300 and 1400 exposures.

RESULTS

After three years of exposure, approximately 10 percent of the blocks remain on the roof in good condition, the only evidences of failure being normal chalking of the pigment. As long as the paint must ultimately fail, chalking is the most desirable type of failure because a chalked surface lends itself most readily to repainting. Most of the other failures were due to blistering and peeling.

CONCLUSIONS

To evaluate these systems more clearly the following tabulations were made showing the results of the various finishing coats over the various primers, the results being expressed as the number of good blocks remaining on the roof. Table 3 compares primer-finishing coat combinations irrespective of color.

TABLE 3

No.	1	2	3	4	5	6	7	8	9	10	11	No Prime	Total
A	18	16	18	8	2	4	0	2	0	0	0	8	76
B	12	12	6	2	2	0	2	0	0	0	0	6	42
C	14	8	6	4	0	0	0	0	0	0	0	12	44
D	14	14	2	6	2	2	2	0	0	0	0	8	50
E	12	16	12	4	4	2	4	0	0	0	0	6	60
Totals	70	66	44	24	10	8	8	2	0	0	0	40	272

Examination of Table 3 shows:

(1) Primers No. 1 and 2 are outstanding in performance, and No. 1, the long oil type, has a slight edge over No. 2, the Phenolic Varnish A —aluminum bronze type. At first thought one would expect No. 2 to be superior because of its greater alkali resistance than the oil type. However, this greater alkali resistance may have been counteracted by the fact that the films deposited were less flexible than No. 1 and therefore more susceptible to damage by thermal changes and other weathering effects.

(2) Primers No. 3, 5, and 6, although of similar vehicle composition to No. 2, did not stand up nearly as well as No. 2. While some difference could be expected between similar resins from different sources of supply, that fact alone cannot account for the vast differences existing here. We must therefore give credit to the Aluminum Bronze as being a superior primer pigment.

(3) That it is better to have no primer at all rather than a poor primer, as it is shown that there are more good blocks remaining without primer than there are with primers No. 5 to 11 combined.

(4) That the straight linseed oil finishing coats gave a good account of themselves, and lead all others in performance. Again, this is probably due to the fact that the modified paints are less flexible and therefore less resistant to weathering. It is reasonable to suppose that over suitable primers, high alkali resistance in the topcoats is neither necessary nor desirable if it is obtained at the expense of flexibility. It is significant ,however, that the topcoat that gave the best account of itself as a primer is phenol modified.

(5) That the difference in performance between all finishing coats is not great, and that the primer, being the first line of defense against attacks of alkali and moisture from within, is the most important part of any system for finishing concrete.

Let us now take a look at the effect of various types of pigmentation, and inasmuch as we have shown that the nature of the finishing coat is less important than that of the primer, let us make the pigment comparisons with the various primers.

TABLE 4

No.	1	2	3	4	5	6	7	8	9	10	11	No. Prime	Total
Toluidine Red	10	10	8			2							30
White Lead Zinc Oxide	6	8	6	2	2							4	28
Iron Blue Tint	4	4			2								10
Ultra Blue Tint	4	4	2									2	12
Chrome Green	10	8	4	6			2					10	40
Red Oxide	18	18	12	6		4	6					8	72
White Lead TiO^2	14	12	8	8	6	2		2				16	68
Chromium Oxide	2	2	2										6
Yellow	2		2	2									6
Total	70	66	44	24	10	8	8	2	0	0	0	40	272

The results shown in Table 4 are not startling. As a matter of fact, they are in line with the behavior exhibited by these same pigments

when used in other types of vehicles and applied to other types of surfaces for exterior exposure. They tell us, however, that the painting of concrete, (again, over a suitable primer), offers no particular pigmentation problems other than those ordinarily encountered, and calls for no special treatment.

In this, as in most other investigations, involving painting, there have been some results for which we cannot account. Some of the results did not check. It is a well known fact that the film thickness of any paint film is an important function in its durability. While an effort was made to apply the paints as uniformly as possible, we cannot afford to neglect the human element in the application, which is undoubtedly responsible for some of the conflicting results, and, of course, this same human element participated in the manufacture of the blocks.

We have not attempted to use any of the data except that which is logical and which can be explained by a reasonable formulating theory, and it is that data, and only that, which has been presented above as the basis for our conclusions.

The results obtained in this test have suggested other combinations of primer and finish coat vehicles to be tried, and while we feel that our results have been highly gratifying, it is our intention to continue the work.

An expression of appreciation is due to the Portland Cement Association for its excellent cooperation on this problem and to Carl J. Borger of the Ault and Wiborg Corp. for his tireless efforts in carrying on the research and assembling the data.

Discussion, to close in the September Supplement, should reach the A. C. I. Secretary by July 20.

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Discussion of a paper by Paul O. Blackmore:

Paint for Durability on Concrete Surfaces*

BY E. F. HICKSON†

There are a few questions in reference to Mr. Blackmore's paper, the answers to which would be interesting.

Since primer No. 1 appears to be superior, some information might be given concerning some of its properties, such as pigment by volume, nonvolatile, hiding, sealing, coverage, finish, and drying time.

It would be instructive to know what pigments were used in primers 3, 5 and 6. Since primer No. 2 was one of the best and this improved quality is attributed to aluminum pigment, it would be interesting to know what pigments were used in primers 3, 5 and 6.

It would also be instructive to know the exact composition of the finishing coats in Table 2, particularly Finish A.

We note that types of failures were indicated by blistering, peeling and chalking. Were there no failures by saponification of vehicles or by checking, cracking, color spotting and color fading of the paint coats?

Presumably the sides and backs of the panels were painted in the same manner as the front.

In painting concrete and brick with oil paints, the Paint Section here at the Bureau and the National Paint Association have been recommending for several years that after the surface is suitably primed the same finishing paints that are used on exterior wood may be used on concrete and brick (in other words the usual outside linseed oil house paints). For a primer, we have been suggesting a thinned house paint, viz: to a gallon of a good outside house paint add about ¾ gal. of spar varnish (Federal Specification TT-V-121a) and 1 to 2 pints of turpentine or mineral spirits.

*JOURNAL, Amer. Concrete Inst., June 1939; *Proceedings* Vol. 35, p. 545.
†Chemist, National Bureau of Standards, Washington, D. C.

AUTHOR'S CLOSURE

The No. 1 primer has a pigment volume of 22 per cent and a pigment weight of 64 per cent. It contains 98½ per cent non-volatile matter. The hiding, sealing and coverage are largely functions of the surface characteristics of the material to be painted, the technique of the painter, and the results expected of the paint. Surface characteristics, such as smoothness and porosity obviously influence the spreading characteristics and the resultant mileage of the paint. No. 1 primer, being dark gray in color, will give complete hiding in very thin films, but there is plenty of evidence on record to show that "stretching" the coverage too far generally results in poor protection and early failure. (See Circular No. 586 of the Scientific Section of the National Paint, Varnish and Lacquer Association, Inc., issued July, 1939).

It has also been shown that no two painters will get the same mileage out of a gallon of paint. It is probably safe to say, however, that on a relatively smooth surface, and applied in sufficient film thickness to assure good durability (about 2 miles per coat), No. 1 Primer can be expected to cover from 600 to 700 sq. ft. in one coat. The finish is low enough in lustre to prevent crawling of the succeeding coats. The drying is slow, 48 to 72 hours being the minimum, and it will be recalled that in the painting of the blocks, one week was allowed between primer and finish coats.

In regard to "types of failure," checking and cracking were observed as early stages of peeling. Some of the peeling was probably due to saponification of vehicles, although we feel that the concrete blocks, as prepared, were very uniform in composition and free from "hot spots." Obviously, uniform alkalinity in the blocks would be essential if the results were to be comparative. We did not notice any color spotting. Fading occurred in varying amounts, depending on the types of pigment used, and was comparable to the behavior of house paint of the same color. In other words, the fact that they were applied over concrete did not appear to produce anything abnormal in the fading tendencies of the paints.

The sides and backs of the panels were primed but received no finish coat.

Mr. Hickson's recommendations for painting concrete are quite in line with our findings. (See paper, p. 550, first paragraph). The mixture which he proposes for the primer, using spar varnish, TT-V121a, would yield a product similar to primers 3, 5 and 6.

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Factors Affecting the Resistance to Freezing and Thawing of Vibrated Concrete Made of Crushed Dolomite*

By M. O. Withey†

SYNOPSIS

The relative resistances of a hundred concrete prisms of various proportions to 150 cycles of freezing and thawing is reported. Resistances were based upon comparisons of flexural strengths and compressive strengths of normal and frozen and thawed specimens. Expansion and loss in weight determinations were also made. Little or no damage was suffered by concrete with 0.54 water-cement ratio, placed with ¼-in. slump and carrying 4¾ sacks of cement per cubic yard. Mixes of lower cement content and certain wetter mixes of somewhat higher cement content were less durable. The flexure test was a sensitive measure of the resistance of concrete to freezing and thawing.

INTRODUCTION

How much should the cement content of concrete be to provide a proper resistance to freezing and thawing is a question that has perplexed construction engineers for a long time. An answer to this question was the objective in the tests to be described. Whereas many experiments have been made to ascertain the resistance of concrete to freezing and thawing, the basis of estimating that resistance has usually been loss in weight, expansion during freezing and thawing, or loss in compressive strength due to freezing and thawing. Owing to the frequency of bending stresses in concrete subjected to weathering, pavement and sidewalk slabs for examples, and the sensitivity of the flexural strength to transverse cracks and other deteriorating influences, the bending test is believed to be a better criterion of the damage wrought by weathering than the other measures mentioned. This opinion is the outgrowth of results of some weathering tests on concrete reported to the Highway Research Board in Decem-

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ber 1933 by L. O. Hanson and the work done by the Special Project Committee on Cements and reported to the same body in 1936. It lead to the adoption of 6 by 6 by 18-in. prisms for the flexure and modified cube tests for the program herein reported.

The specimens for the tests were placed by vibration in order to take advantage of the dry consistencies and high densities which are obtainable through such procedure. The tests were made with a highly durable and uniform crushed dolomite from the Niagara formation in Eastern Wisconsin. The program contained variables in cement content, water-cement ratio, and consistency. One hundred one cross-bending tests and more than 200 compression tests, together with some loss in weight and expansion readings taken during the freezing and thawing tests were included.

The tests were begun in the spring of 1935 and extended through that year. They were made possible by contribution of funds from the Wisconsin Alumni Research Foundation and some N. Y. A. student help. Much of the making and testing of the specimens was done by Messrs. L. E. Bidwell and E. K. Neroda. Messrs. J. E. Liebmann and H. H. Radke assisted in preparation of material for this report.

MAKING AND CURING SPECIMENS

A normal portland cement of standard quality was used in these tests. Janesville sand passing a ¼-in. screen, having a fineness modulus of 2.6, and a specific weight when dry and rodded of 111 lb. per cu. ft. was used as fine aggregate. Lannon dolomite crushed to pass a 1½ in. sieve had 48 per cent of its particles retained on a ¾-in. sieve, 90 per cent on a ⅜-in. sieve, and 100 per cent on a ¼-in. screen. It weighed, dry and rodded, 98 lb. per cu. ft. The specific gravities of the materials were: cement 3.13; sand 2.70; and stone 2.73. The stone and sand were air dry and absorbed 0.7 and 0.6 per cent of water, respectively.

Mixing was thoroughly done by hand in galvanized iron trays. Each batch after mixing was allowed to stand ½ hour under damp canvas to permit absorption of water by the aggregate. The slump was then determined and the concrete cast into steel channel molds. Six specimens were made in each mold, the bottoms being lined with asphalt felt to prevent leakage of water from the forms during vibration. Expansion prisms were provided with ¼-in. brass plugs running transversely through the specimens and spaced 16 in. c. to c. These plugs were provided with end holes to permit strain gage measurements.

All specimens of 0-, ¼- and 2-in. slumps were compacted in the molds with an external electric vibrator attached to a 4 by 5 by 16-in.

wood block. The vibrator was operated on the top of each specimen until water appeared at the surface. The speed of the vibrator was 3600 r.p.m. and its amplitude when resting on sponge rubber cushions 6 in. apart was 0.06 in. The specimens of 6-in. slump were hand puddled and surfaced along ends and sides with a trowel.

After two days in molds specimens were weighed in air and immersed in water to determine specific weights and densities. They were then stored in a moist closet until 28 days old. At that age 5 specimens of each water-cement ratio having 0-slump and 3 of every other water-cement ratio and slump were subjected to bending and to compression tests. A like number of each type was subjected to freezing and thawing and a similar group was retained in the moist closet for strength tests to be made at the conclusion of the freezing and thawing program.

METHOD OF TESTING

Prisms for the freezing and thawing test were soaked in water for 48 hours prior to freezing. They were loaded in trays with 6 by 18-in. bottom faces down and partially immersed to a depth of 1-in. in water when placed in the freezer. Fig. 1 shows the type of freezing and thawing cycle as determined from thermometer readings taken at the middle of 2 by 2 by 9-in. mortar prisms subjected to similar conditions in later tests. After 16 hours in the freezer the trucks carrying trays of specimens were withdrawn and the specimens were thawed in lake water at 70° F. which was sprayed over their tops.

In the cross-bending tests the prisms were supported bottoms down over a 16-in. span on spherical-seated knife edges and subjected to center loads at nominal speeds of head of 0.027 in. per min. The fractured portions were then capped on top and bottom surfaces with a neat paste of high early strength cement containing 3 per cent by weight of calcium chloride and allowed to stand over night under wet canvas. On the following day the modified cubes were crushed in a 200,000-lb. three screw Riehle machine operated at a nominal head speed of 0.06 in. per min.

DISCUSSION OF RESULTS

Table 1 shows the proportions and properties of the mixes tested. It will be observed that the water-cement ratios varied from 0.54 to 0.68, by weight, or from 6.1 to 7.6 gal. per sack, the cement contents from approximately 3 to 6¼ sacks per cu. yd., and the specific weights and densities of all mixes were high.

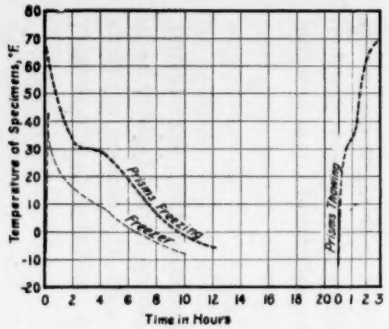

FIG. 1 — AVERAGE RATE OF FREEZING AND THAWING OF 2 X 2 X 9-IN. MORTAR PRISMS

TABLE 1—PROPERTIES OF LANNON DOLOMITE CONCRETE MIXES

Specimens	Mix, by Weight C : S : D	Slump, in.	Specific Weight, lb./cu. ft.	w/c by Weight	c	p	c/v	Cement
C1-15	1 : 4.60 : 8.59	0	154.6	0.675	0.0530	0.8570	0.371	2.97
C21-29	1 : 3.55 : 6.62	¼	155.0	0.675	0.0667	0.8460	0.433	3.73
C31-39	1 : 2.98 : 5.55	2	154.3	0.675	0.0771	0.8335	0.463	4.32
C41-49	1 : 2.56 : 4.77	6	154.2	0.675	0.0873	0.8232	0.494	4.89
F1-15	1 : 4.05 : 7.52	0	155.6	0.616	0.0601	0.8601	0.430	3.36
F21-29	1 : 3.18 : 5.93	¼	155.2	0.616	0.0739	0.8465	0.481	4.14
I1-15	1 : 3.49 : 6.50	0	156.5	0.54	0.0692	0.8633	0.508	3.87
I21-30	1 : 2.76 : 5.13	½	155.7	0.54	0.0843	0.8479	0.554	4.72
I31-39	1 : 2.34 : 4.35	2	154.6	0.54	0.0958	0.9325	0.572	5.37
I41-53	1 : 1.95 : 3.62	6	154.3	0.54	0.1117	0.8192	0.618	6.26

TABLE 2—DOLOMITE CONCRETE STRENGTHS BEFORE AND AFTER FREEZING

Specimens	Mix by Weight C : S : D	Slump, in.	w/c by Weight	Strength in Pounds Per Square Inch: Modulus of Rupture 28-Day	Modulus of Rupture 3-Mo. Estimated (N)	Modulus of Rupture 7-Mo.	Modulus of Rupture After Freezing 150 Cycles (F)	Modulus of Rupture Ratio F/N	Compressive Strength 28-Day	Compressive Strength 3-Mo. Estimated (N)	Compressive Strength 7-Mo.	Compressive Strength After Freezing 150 Cycles (F)	Compressive Strength Ratio F/N
C1-15	1 : 4.60 : 8.59	0	0.675	477	563	627	329	0.58	3538	4150	4610	4115	0.99
C21-29	1 : 3.55 : 6.62	¼	0.675	518	663	772	319	0.49	3589	4300	4845	4045	0.94
C31-39	1 : 2.98 : 5.55	2	0.675	582	705	798	411	0.58	3352	4010	4510	4125	1.03
C41-49	1 : 2.56 : 4.77	6	0.675	614	758	867	239	0.32	3548	4480	5185	3810	0.85
F1-15	1 : 4.05 : 7.52	0	0.616	602	698	770	627	0.90	4208	5060	5700	4850	0.95
F21-29	1 : 3.18 : 5.93	¼	0.616	616	723	804	523	0.72	3902	4789	5435	4470	0.94
I1-15	1 : 3.49 : 6.50	0	0.54	730	846	934	773	0.91	4954	5520	5930	5560	1.01
I21-30	1 : 2.76 : 5.13	¼	0.54	651	823	952	844	1.02	4587	5235	5715	5380	1.03
I31-39	1 : 2.34 : 4.35	2	0.54	700	921	1087	777	0.84	4417	4960	5365	5090	1.02
I41-53	1 : 1.95 : 3.62	6	0.54	758	966	1130	879	0.91	4891	5535	6020	5715	1.03

Normal strengths and strengths after 150 cycles of freezing and thawing are given in Table 2. The estimated strengths at three months are considered to be the strengths which the frozen and thawed specimens would have attained due to the periods of water soaking after thawing provided no deterioration occurred during the 150 cycles of freezing. They were estimated upon the basis that the strength would increase from 1 to 7 months in proportion to the logarithm of the age. Hence $S_3 = 0.43\ S_1 + 0.57\ S_7$, where S_1, S_3 and S_7 are the strengths at 1, 3 and 7 months, respectively. The ratios of F/N based on this estimated strength furnish an index of the damage done due to the freezing and thawing test. The F/N ratios in Table 2 coupled with the diagrams of Fig. 2a show very clearly that the modulus of rupture after freezing, or the F/N ratio based on it, is a more sensitive measure of the damage done by freezing and thawing than is the compressive strength, or the F/N ratio based on it.

The diagrams in Fig. 2 indicate that the water-cement ratio, Fig. 2(a), is a better single index of resistance to freezing and thawing than the cement-content Fig. 2(b). The latter figure shows that strength-cement-content relationship is dependent upon constancy of the consistency. Curves similar to those in Fig. 2b can also be plotted between the strengths and the cement-voids ratios, c/v.

The data in Table 3 and the curves of Fig. 3 show the expansion and loss in weight data based on averages of 2 or 5 specimens per point. From Fig. 3(a) and the strength data it appears that the expansion-w/c curve is a more discriminating index of the damage done by freezing and thawing than the loss in weight.

The relative damage done by 150 cycles of freezing and thawing to concrete of different water-cement ratios is well shown in Fig. 4. None of the specimens disintegrated in these tests but the frozen and

TABLE 3—EXPANSION AND LOSS IN WEIGHT OF DOLOMITE CONCRETE DUE TO FREEZING AND THAWING

Specimens	Mix by Weight C : S : D	Slump, in.	w/c by Weight	Absorption After 7 Mo. in Moist Room, Per Cent	After 150 Cycles of Freezing and Thawing	
					Per Cent Loss in Weight	Per Cent Expansion
C1-15	1 : 4.60 : 8.59	0	0.675	1.03	1.21	0.077
C21-29	1 : 3.55 : 6.62	¼	0.675	0.89	0.76	
C31-39	1 : 2.98 : 5.55	2	0.675	0.98	1.79	
C41-49	1 : 2.56 : 4.77	6	0.675	1.23	3.77	
F1-15	1 : 4.05 : 7.52	0	0.616	0.98	0.50	0.0265
F21-29	1 : 3.18 : 5.93	¼	0.616	0.96	0.50	
I1-15	1 : 3.49 : 6.50	0	0.54	1.13	0.47	0.0142
I21-30	1 : 2.76 : 5.13	¼	0.54	0.87	0.33	0.0265
I31-39	1 : 2.34 : 4.35	2	0.54	1.17	0.33	
I41-53	1 : 1.95 : 3.62	6	0.54	1.23	0.37	0.0230

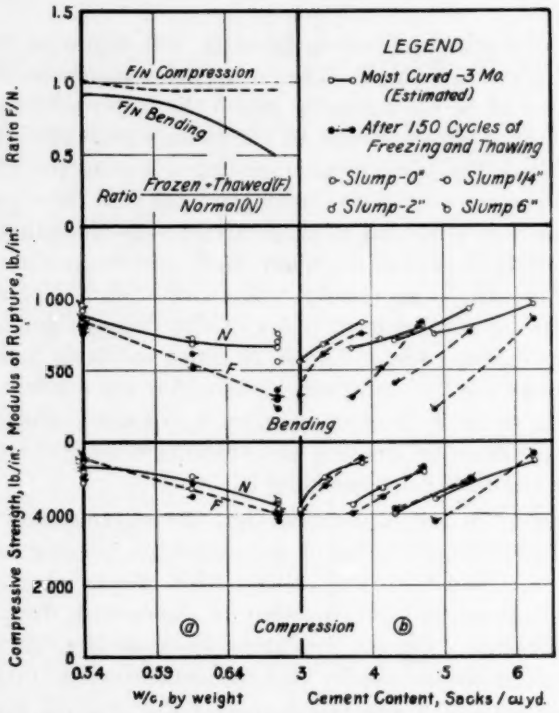

FIG. 2—EFFECT OF FREEZING AND THAWING ON THE STRENGTH OF VIBRATED CONCRETE MADE OF CRUSHED DOLOMITE

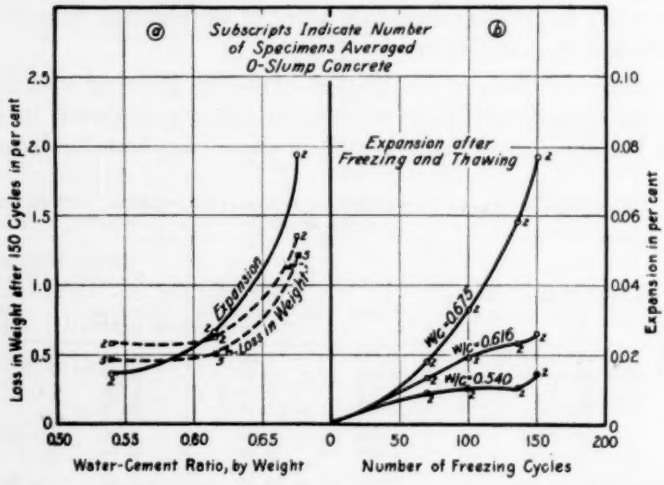

FIG. 3—THE INFLUENCE OF FREEZING AND THAWING ON LOSS IN WEIGHT AND EXPANSION OF VIBRATED CONCRETE MADE OF CRUSHED DOLOMITE

thawed C-specimens (w/c = 0.68) exhibited lower strengths, much greater variability in strength, and greater losses in strength due to freezing and thawing than any other group of specimens. As would be expected the I-specimens (w/c = 0.54) exhibited the best resistance of the mixes tried. Of the consistencies used, the ¼-in. slump appears to have been the most effective under the conditions of placing used in these tests.

From comparison of the results of compressive tests on gravel concrete* made after 150 cycles of freezing and thawing and the data herein recorded, it appears that the concrete of crushed dolomite was more resistant to freezing and thawing than gravel concrete mixes of like w/c ratios.

CONCLUSIONS

Assuming uniform, durable, well-graded aggregate less than 1½ in. in diameter, cement of standard quality, and proper conditions of mixing, placing, and curing the following conclusions appear to be justified.

1. Concrete with a water-cement ratio of 0.54 will possess good resistance to freezing and thawing.

2. With water-cement ratio constant, the tests failed to disclose marked differences in resistance to freezing and thawing due to changes in consistency from 0 to 6-in. slump.

3. Since economy demands that the cement content be reduced as low as is consistent with efficient handling, placing and finishing, the amount of mixing water should be made as small as practicable. Where a ¼-in. slump can be used effectively, concrete of good resistance to freezing and thawing can be secured with a cement content of 4¾ sacks per cu. yd. For conditions of placement where a slump of 6-in. is demanded good resistance can be obtained by raising the cement content to 6¼ sacks per cu. yd. The use of the drier, denser, and leaner mixes is advocated wherever they are suitable.

4. Concrete of 0.54 water-cement ratio having good resistance to freezing and thawing should exhibit an average 28-day crushing strength of 4200 p.s.i. and a modulus of rupture of 625 p.s.i.

5. Mixes of 0-slump containing less than 4 sacks of cement per cu. yd. or mixes of greater slump than ¼-in. containing less than 5 sacks per cu. yd. showed a lack of flexural strength after 150 cycles of freezing and thawing and in many instances visible damage at edges and corners.

*JOURNAL Amer. Concrete Inst., May-June, 1935; *Proceedings* Vol. 31, p. 528.

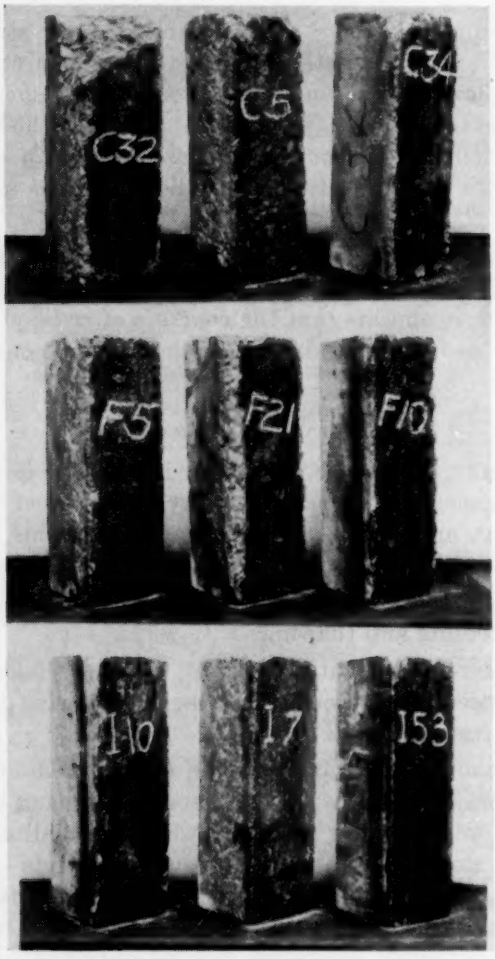

FIG. 4—CONDITION OF SPECIMENS AFTER 150 CYCLES OF FREEZING AND THAWING

Specimens C5-34, W/C = 0.68; F5-21, W/C = 0.62; I7-53, W/C = 0.54.

6. The flexure test is a far more discriminating and sensitive criterion of resistance to freezing and thawing than the other tests used in this program.

7. The utility of vibration as a means of placing effectively concrete of dry consistency was again demonstrated in these experiments.

Discussion, to close in the September Supplement, should reach the A. C. I. Secretary by July 20.

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Durability of Concrete Pavement—Experiences in New York State*

By E. C. Lawton†

SYNOPSIS

This paper presents case histories of a number of pavements involving matters of geography, exposure, aggregates and cements under various conditions of construction and use and with varying results in durability under traffic and exposure. The first record is of a pavement in 1925. New York State's experience, especially with its 50 odd experimental projects, has led to (1) standard construction employing one bag of a brand of natural cement to six bags of portland in all pavement mixes; (2) a tricalcium aluminate limit of 9 per cent and a tricalcium silicate limit of 55 per cent.

The question of durability of concrete, particularly of cement concrete pavement, is not new to the American Concrete Institute or to highway engineers generally but the most difficult part of the problem is to decide on a measuring stick for "durability." In our State we generally depend on what we call the "Service Record" of the pavement or project, or parts of projects, as compared with other pavements which have a longer or the longest useful life under traffic. To make these comparisons we make careful inspections of the condition of concrete pavements, give them a service record rating, such as A, B, and C, and then study the materials which were used in their construction. Our studies have to do largely with the properties and tests of aggregates—cement, fine aggregate and coarse aggregate—that were used in these classes or groups of service records of pavements.

Our first study in field performance began in the late twenties and early thirties and was largely devoted to sands. Back in 1932 we presented a paper—"Progress in Determining Suitability of Concrete Pavement Aggregates," before the Association of Highway Officials

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of the North Atlantic States. There is really no difference between the use of "durability" in this paper and "suitability" in the paper before the North Atlantic States. The objective is the same. The scope of our experiences in New York State is now so great and covers such a period that this paper becomes an historical resume of what we have done, from time to time, and a brief discussion of our observations on a number of experimental sections embodying different combinations and different qualities of aggregates in concrete pavement throughout the State. Our comments upon different behaviours will describe briefly the aggregates and reasons for their use and our opinions, if any, will be based upon the service record of a number of experimental sections. Of course the words "service record" are hard to define but we consider it is measured largely by the absence or presence of scale, the absence or presence of appreciable disintegration of the exposed wearing surfaces, and those pavements which have disintegrated so far as to make it necessary to surface treat or place bituminous tops on them to prolong their useful life.

It is our observation that proper application of surface treatments or the construction of heavy bituminous tops will unquestionably prolong the useful life of doubtful concrete pavement. These methods prevent direct attacks on concrete from the atmosphere, from tire chains, and from calcium or sodium chloride salts used in removing ice from the pavements. Scaling first affects the mortar in the concrete. The mortar is composed of sand and cement. Therefore, it is fair to assume that one or both of these ingredients are contributing causes to scaling and disintegration.

SAND REQUIREMENTS

In our study of the behaviour of sands (*Proceedings* Association of Highway Officials of the North Atlantic States, 1932) we dwelt upon the qualities and behavior of types of sand commercially available in our state and methods of testing them. At that time and at the present time sands are generally accepted or rejected, taking into consideration the properties on the following Typical Report shown in Table 1*.

We refer to this report as a "rational analysis" which determines the various percentages of kaolin, feldspar, quartz, calcium carbonates, magnesium carbonates, and oxides. Many supposed particles of sand consist of further minute grains (generally silica) held together by kaolin or clay which has some cementing value. When this kaolin

*This table is from a diagram of experimental sections. This and similar diagrams for other experimental roads discussed in this paper are not reproduced.—EDITOR

TABLE 1—TYPICAL REPORT

Sand No. 1		*Sand No. 2*	
Kaolin	25.8	Kaolin	3.01
Feldspar	17.5	Feldspar	16.22
Quartz	44.7	Quartz	78.28
$CaCO_3$	1.0	$CaCO_3$	0.25
$MgCO_3$	2.1	$MgCO_3$	0.24
R_2O_3	8.9	R_2O_3	2.00
Abrasion	30.5	Abrasion	12.6
Loam and Silt	3.8	Loam and Silt	2.1
Specific Gravity	2.64	Specific Gravity	2.63
Voids	34.9	Voids	33.9
F. M.	3.48	F. M.	3.26
Compression		Compression	
7 day	103	7 day	136
28 day	115	28 day	170
Tension		Tension	
7 day	143	7 day	126
28 day	131	28 day	118
Organic	O.K.	Organic	O.K.
5 Cycl. Sodium Sulphate	12.1%	5 Cycl. Sodium Sulphate	1.7%

or clay is broken up, miniature disintegration of sand grains takes place. Hence, disintegration of mortar may occur. Therefore, the percentage of kaolin should be kept to a minimum. When magnesium carbonate occurs as water lime, it should be excluded or kept to a minimum. The most desirable components are feldspar, quartz, limestone ($CaCO_3$) and oxides of aluminum and iron (R_2O_3). Our specification for sand requires a certain gradation and that all sand must be thoroughly washed before using. When subjected to 10 cycles of the sodium sulphate soundness test, it shall lose not more than 10 per cent by weight. In a certain doubtful region of the State (consisting of two districts) we have a further requirement that "fine aggregate used in the top 2½ in. of concrete pavement shall show not more than 6 per cent of kaolin nor less than 75 per cent of Quartz plus Feldspar, as shown by rational analysis."

Many of our sands absorb ten times the amount of moisture that others will absorb. It is a well-known fact that sands high in silica, feldspar, and calcium carbonate will absorb practically no moisture. Sands, however, which contain such unsound elements as shale, argillaceous sandstone and limestone, chert, etc., are relatively soft elements and will absorb considerable moisture. When moisture is in the mortar and subjected to alternate freezing and thawing, disruptive forces are doing work which causes first the symptom of scaling and then probably the disease of disintegration. It seems obvious, therefore, that we should prevent the absorption of moisture by the sand particles themselves and the permeability of moisture into minute voids of the mortar itself in order to prevent or retard scaling or disintegration as long as possible. The so-called rational analysis of sands is, therefore, a very valuable guide in passing upon the desirable or undesirable properties of sand itself. I have dwelt

upon the rational analysis of sand at some length as we feel that a great many poor sands were rejected throughout the State by virtue of our first undertaking. We felt then, and we feel now, that we largely overcame such scaling as could be attributed to sand. We feel also that comparisons of tension or compression tests of mortar or concrete are not a reliable yardstick of durability. At that time also we turned out investigations to cements, in addition to other aggregates, and the relation of cement to scaling and disintegration which will be discussed later in this paper.

The problem of scaling in 1925 was particularly acute in the western part of the state and we constructed an experimental road, known as the Lawtons-Gowanda road (C. H. 525) in southern Erie County. This was a little prior to our extensive sand studies. There was some thought (and there still is) that coarse aggregate might be the culprit.

LAWTONS-GOWANDA-ROAD

In this project, built in 1924-25, we used the same sand and cement throughout with three different types of coarse aggregate, commercially available in the Buffalo areas. The purpose of this selection was to make a comparison of the behaviour of coarse aggregate in the western part of the State which was not blessed generally with good coarse or fine aggregate.

Approximately one-third of the project was constructed with gravel; approximately one-third with stone, and approximately one-third with slag as coarse aggregate. The pavement consisted of two 9-foot lanes, 7½ in. thick at the edge and 6 in. at the center. It was reinforced with what we call marginal bar reinforcement. Expansion joints are about ⅜ in. thick at 40-ft. intervals. In 1926 it carried a mixed traffic of about 1400 vehicles in 12 hours. In 1938 it carried a mixed traffic of 2400 vehicles in 12 hours. This was a reconstruction project and the new concrete pavement was built largely on old macadam pavement and embankment which was well compacted and stabilized and reasonably free of so-called foundation trouble. In other words, it has adequate structura. strength for traffic. Any surface defects, therefore, could be attributed to different behaviour of course aggregates rather than inadequate design. Table 2 shows the average gradation of the fine and coarse aggregate and data concerning test cubes and cores.

The general pavement mix for gravel and stone aggregate was 1:1½:3. However, it was necessary to increase the volume of sand and to use smaller sizes of slag than stone or gravel to obtain proper workability and finish. We found the best mix to be 1:2:3, using

TABLE 2—LAWTONS-GOWANDA ROAD

RESULTS OF CORE TESTS

Station	Depth	Mix	Lbs. per Sq. In. Corr. for $h/d = 2$	Aggregate
238+83	6 11/16"	1:1½:3	4460	Stone
215+60	7 1/16"	1:1½:3	4540	Stone
199+68	7 3/16"	1:1½:3	4170	Stone
215+60	7 1/16"	1:1½:3	3960	Slag
199+95	7"	1:1½:3	3860	Slag
236+80	7 11/16"	1:1½:3	3640	Slag

MATERIAL SIZES

Sand—87% pass ¼"—60% pass 20—10% pass 50—2% loam
Slag—100% pass 2¾"—95% pass 1½"—20% pass ⅝"
Stone—100% pass 2¾"—50-60% pass 1½"—0% pass ⅝"
Gravel—100% pass 2¾"—70% pass 1½"—4% pass ⅝"

COMPRESSION TESTS ON CUBES MADE IN FIELD DURING PROGRESS OF THE WORK

Aggregate	Station	Mix	Age 28-30 Dys.	Age 42 Dys.	Age 58-62 Dys.	Age 69-76 Dys.	Age 97-105 Dys.
Stone	20-29 L.	1:1½:3	.....	.....	.....	.....	4560–5280 (105) (Av. 4920)
Stone	278-289 R.	1:1½:3	.....	3360–3660 (42)	.....	.....	.....
Stone	259-269 R.	1:1½:3	.....	3830–3830 (42) (Av. 3670)	.....	.....	.....
Stone	203-212 R.	1:1½:3	3200–3330 (28)				
Stone	220-230 R.	1:1½:3	3000–3720 (28)				
Stone	243-253 R.	1:1½:3	3880–3880 (28)	.....	.....	.....	.....
Stone	302-313 R.	1:1½:3	3275–3360 (30) (Av. 3455)	.....	.....	.....	.....
Gravel	13-20 R.	1:1½:3	.....	.....	.....	.....	1950–4310 (99) (Av. 3130)
Gravel	28-60 R.	1:1½:3	.....	.....	.....	3280–3340 (69)	.....
Gravel	157-163 L.	1:1½:3	.....	.....	.....	2170–2640 (76) (Av. 2850)	.....
Gravel	126-128 R.	1:1½:3	.....	.....	1390–3390 (60)	.....	.....
Gravel	144-155 R.	1:1½:3	.....	.....	3280–4610 (58)	.....	.....
Gravel	100-116 R.	1:1½:3	.....	.....	2700–3840 (59)	.....	.....
Gravel	86-89 R.	1:1½:3	.....	.....	3360–3440 (62) (Av. 3250)	.....	.....
Slag	139-146 L.	1:2:3	.....	.....	.....	.....	4080–4720 (97) (Av. 4400)
Slag	304-315 L.	1:1¾:3	.....	3360–3635 (42) (Av. 3500)	.....	.....	.....
Slag	278-291 L.	1:1¾:3	2940–3170 (28)	.....	.....	.....	.....
Slag	257-265 L.	1:2:3	3060–3275 (28)	.....	.....	.....	.....
Slag	207-219 L.	1:1½:3	3060–3880 (30) (Av. 3230)	.....	.....	.....	.....

about 20 per cent No. 1 aggregate (¼ to ⅝ in.) and 80 per cent No. 2 aggregate (⅝ to 1¼) in. The finish of the slag concrete was perhaps more meticulous and the workmanship more of the expert type than was necessary on gravel or stone. The slag pavement now has had a service life of approximately 15 years and is now in reasonably good condition with no serious indication of incipient failure. Cross cracks average three per 40-ft. slab but they are not objectionable; they require little maintenance, and are not as numerous as on many other roads. The pavement appearance indicates an expected useful life of at least five years more before resorting to cover. Compression tests of both cubes and cores indicate that stone had higher strengths than gravel or slag. The latter were about equal. Since then three stone cores, three gravel cores, and seven slag cores have been taken at approximately the same location during the years 1932, 1934, and 1938, with the following average results:

	1932	1934	1938
Slag (7 cores—each year)	5317	6485	6503
Gravel (3 cores—each year)	4107	7107	6970
Stone (3 cores—each year)	5677	6358	6278

I will not attempt to follow this project in detail but will summarize briefly: Nearly all of the stone aggregate has been surface treated and now has a bituminous wearing surface over most of its area. Gravel aggregate is now in fair condition and has retained its original surface texture in places and in other places it is rather deeply scaled In other words, the stone required the first emergency treatment. Up to this time nothing unusual or extraordinary has been done on the slag portions of the pavement. The surface appearance of the slag is still good and appears to have lost only a thin film from the top but there are no evidences of what we commonly call scale of the progressive type. Both the gravel and slag now have a good rating and it will require a little more time to show whether or not one is distinctly better than the other. Incidentally, the sand used in this project has since been rejected for use in cement concrete pavement on account of its soft particles.

In our search for durability in a region not blessed with the best of aggregates, we turned thoughts to two-course construction and built the—

DUNKIRK-SILVER CREEK ROAD

The top course of most of it was 2 in. thick with a 1:1½:2 mix using a No. 1 size gravel, commonly called *pea gravel*, and commercially available in that district; the bottom course 6 in. thick with a

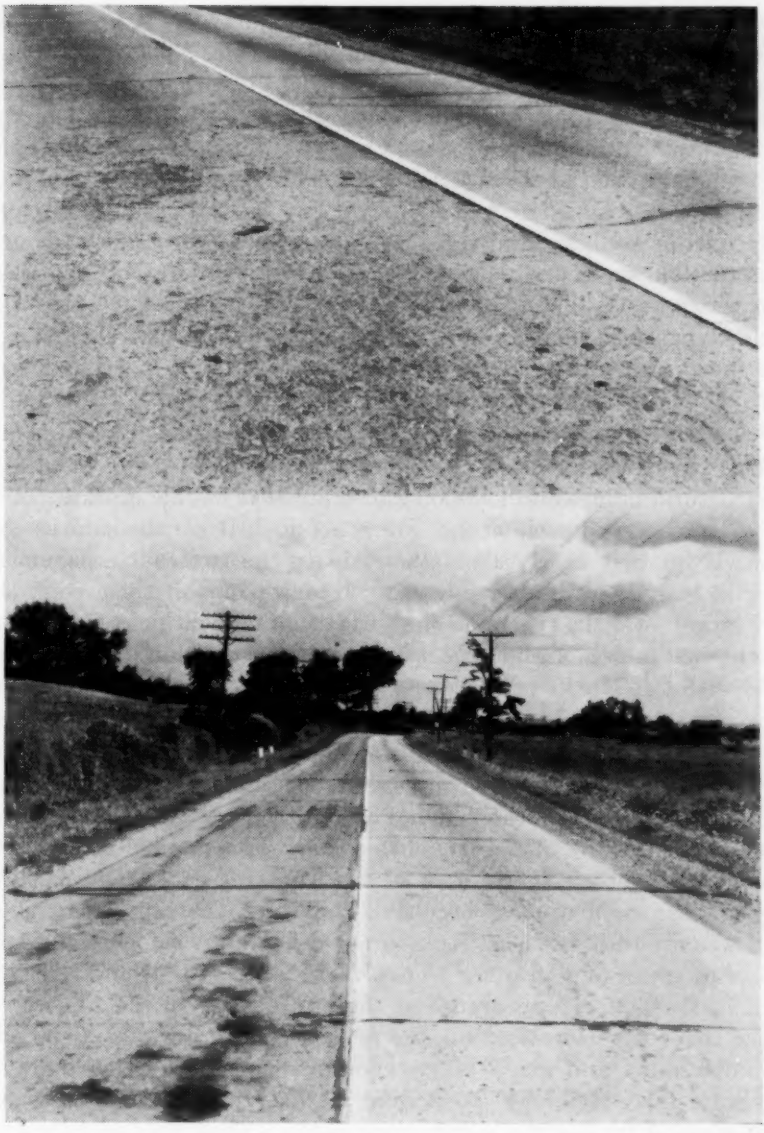

FIG. 1—GRAVEL AGGREGATE CONCRETE ON LEFT; SLAG ON RIGHT

Highway 525 (Lawtons-Gowanda Road), 8/24/38; view southwest from Station 9 ft. east of Station 130+45.

FIG. 2—STONE AGGREGATE PAVEMENT ON LEFT; SLAG ON RIGHT

Highway 525 (Lawtons-Gowanda Road) 8/24/38; view south from Station 314+80.

1:2:3½ mix of broken stone, screened gravel, or slag. A portion (one lane about two miles long) was built monolithic. This project is a 20-ft. concrete pavement, 8-in. uniform depth, with normal bar mat reinforcement and it is 7.65 miles long. It was built between Dunkirk and Silver Creek in Chautauqua County during the years 1928 and 1929 upon an old but well-consolidated county road. The foundation was reasonably adequate and the pavement has not yet been appreciably overstressed by heavy loads. The theory back of two-course construction was to obtain a dense, impervious top of high-class aggregates and utilize local material for bottom course. It was also decided to use a number of different brands of normal cement and one so-called "super cement." The principal brands of cement used were Universal, Bessemer normal cement, very short stretches of Bessemer Super cement, and Lawrence Strongfast cement.

This pavement has been inspected annually for evidence of surface or structural failure and location and number of visible cracks. In so far as transverse visible cracks are concerned, it can be said at this time (1939) that nearly all of the slabs in the two-course pavement have at least two transverse cracks per slab (some of them considerably more). On the two miles (half width) of monolithic construction, which was built alongside the two-course construction, there are no slabs with two cracks per slab and most of them show no evidence of transverse cracks.

In the monolithic construction, which is the easterly lane, adjoining the city of Dunkirk, we used three different sands, two of which have since been rejected by the rational analysis for use in concrete pavement. A large part of the monolithic concrete is in good to excellent condition (except a short piece of the Dunkirk end where calcium chloride has been applied excessively) but the portion where good sand was used has the least scale or no scale, and it has no transverse cracks to speak of. In other words, the quality of the sand reflected itself in the surface appearance of the pavement. In the top course of the two-course construction, the pea gravel aggregate came largely from one source and was of fair to good quality. With this pea gravel aggregate three different sands were used, of which two have since been rejected for use in concrete pavement. (They are the same two sands mentioned in the discussion of the monolithic construction.) While there is some scaling on parts of the two course construction, here again the portions in which good sand was used have a better appearance than where a poor sand may have been used.

In the bottom course of the two-course construction, both slag and gravel were used as coarse aggregate and three different kinds of

sand were used as fine aggregate. These are the same sands mentioned above, two of which have since been rejected for use in concrete pavement. The only visible difference on the surface as between the different coarse aggregates used in the bottom course is in the number of transverse cracks in the slab. There are more transverse cracks per slab where slag was used than where gravel was used in the bottom course. However, there are still far more cracks even where gravel was used in the bottom course than where monolithic construction was used. This could be summed up by saying there are a greater number of transverse cracks per unit of length where two-course construction is used than where one course or the monolithic type of construction is used. The tendency to many transverse cracks in two-course construction is evident on a large mileage of two-course cement concrete pavement in this area.

When this project was constructed we hoped to learn something about the behaviour of different cements but as we have since concluded that some of the sands and aggregates were poor, we may not obtain much helpful information about the cement. While there is some difference in the appearance of different parts of the surface where different cements were used, the principal trend shown by this experimental section is that we may expect more transverse cracks per unit of length in two-course construction than in one-course construction and less scale with good sand than with poor sand (rational analysis).

This project is now only ten years old and shows some scale and some symptoms of scale and has some thin bituminous patches in spots. What scaling there is seems to be progressing outwardly from the joints and at the rate of from eight to ten inches per year. This may be due to overloads as load-transfer joints were not then used by the state. This pavement seems to have reached a critical stage and probably will show a more definite and distinct trend in the next five years than is so far readily apparent. Any definite conclusions, with the possible exception of frequency of transverse cracks, should be deferred.

A 12-hour traffic count on this highway is as follows:

Total 12 Hr.	Heavy Trucks	Percentage of Trucks	Year
3109	211	6.8	1936
4167	303	7.3	1937
3121	201	6.4	1938

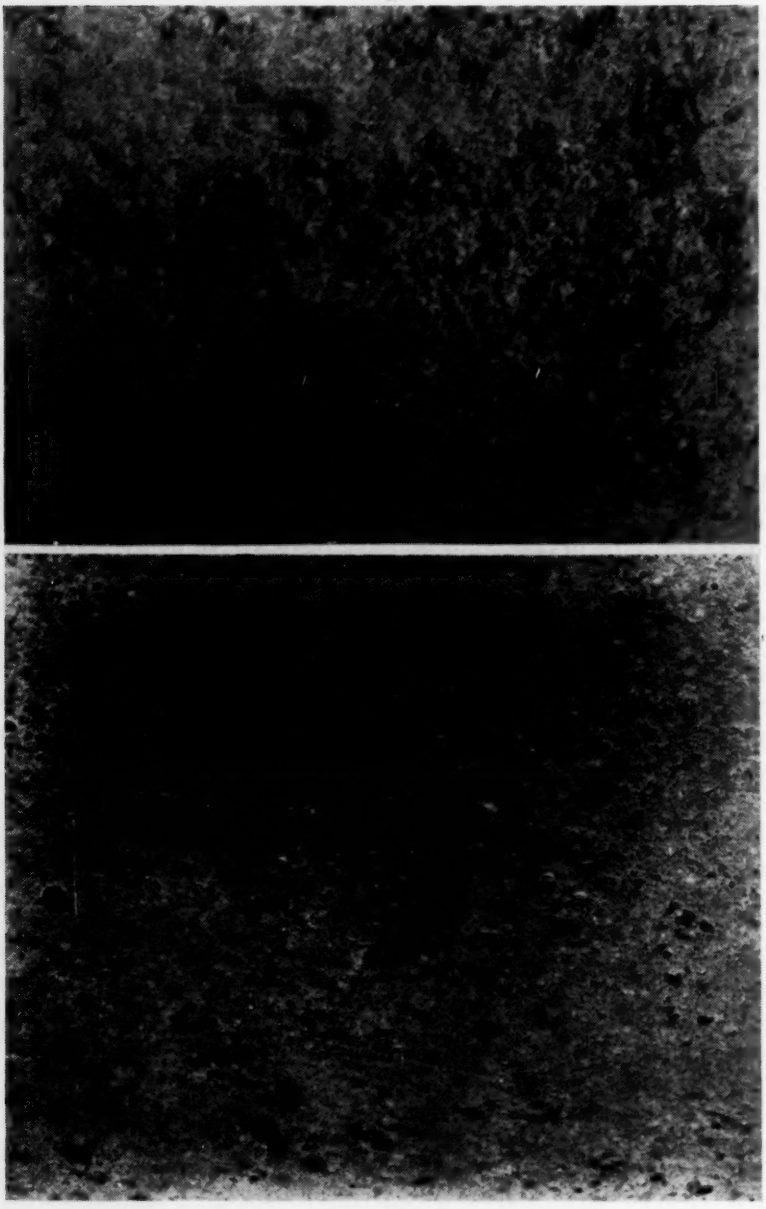

FIG. 3—DETAIL OF GRAVEL AGGREGATE CONCRETE

FIG. 4—DETAIL OF SLAG AGGREGATE CONCRETE

Both at Station 130+45, Highway 525 (Lawtons-Gowanda Road).

CEMENTS

Since our extensive investigation of sand in 1931-32, we have carried on a rather intensive investigation of behaviour of different brands and types of cement. Sometime in 1934 we examined 443 concrete road projects and gave them a service rating. These projects were in four districts of the state and were those which had been constructed prior to 1928. They were examined in the field and were classified according to the service they rendered. Classification was divided into three groups, viz., A—Good, B—Fair, and C—Poor. A pavement which had lasted fifteen years but which had disintegrated to such an extent as to require covering or oiling was placed in the B class with two subdivisions—"B+" ann "B−". It was rated as "B+" if it lasted over 15 years and "B−" if it lasted more than eight and less than twelve years. A road which lasted 20 years before covering became necessary, due to disintegration, was given an A rating. From 15 to 20 years it was given an "A−" rating (Lawtons-Gowanda, for instance) and over 20 years an "A+" rating. A failure, which required covering the pavement either with light surface treatment or heavy material before eight years, was placed in the C class. We considered failures due to disintegration but *not* due to over-loading of light pavements with heavy loads and this distinction is every important.

We then arranged in tabular form all concrete pavements in the A and B groups. From field and laboratory records we traced the origin of cement used in the different projects. The projects were then arranged vertically, according to the ascending percentages of tricalcium aluminate (C_3A). Whe then ascertained the varying computed percentages of tricalcium silicate in the different cements and arranged this data horizontally. One thought we had in mind was to discover, if possible, whether there was a consistent relationship between "service records" and one or both of these properties. At that time there were at least two opinions concerning desirable cement properties. Opinion No. 1 was that high tricalcium silicates were less durable than low tricalcium silicates. Opinion No. 2 was to the effect that cements with high tricalcium aluminates were less durable than those with low tricalcium aluminates. Following our study, we reached a definite conclusion that it is very desirable to keep tricalcium aluminate *as low* as practicable and a tentative conclusion that as tricalcium aluminate increases the tricalcium silicate should decrease. As our conclusions reached, regarding the study of the above data, were highly controversial, and perhaps remain so, and as I was allowed the latitude from the title of this paper to discuss "Experiences in New

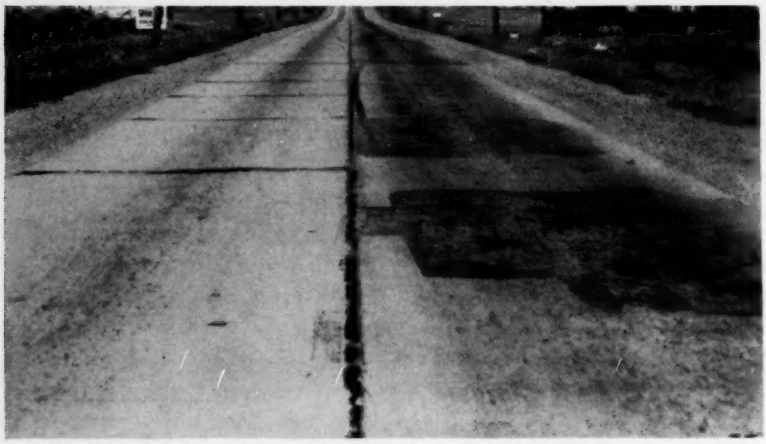

FIG. 5—LEFT LANE IS TWO COURSE PAVEMENT (BOTTOM COURSE SCREEDED); RIGHT LANE ONE COURSE (MONOLITHIC)
Highway 8304 (Dunkirk-Silver Creek Road), 8/23/'38; Station 2+20.

York State," I do not consider it appropriate to include the tabulation in this paper. I have briefly described it, however, to point out the method of making the study and analysis as a sort of prelude to some of the things that follow this preliminary investigation.

The first consequence of this study was that in our specification adopted Jan. 2, 1935, we included a cement provision that "the tricalcium aluminate shall not exceed 9 per cent and the tricalcium silicate shall not exceed 55 per cent." Soon thereafter and in order to continue our field research work, we required and specified, for experimental purposes, cement having different amounts of tricalcium aluminate and tricalcium silicate on many projects throughout the state. (See Appendix for partial list.) For example, a cement designated as 935 means that the tricalcium aluminate is limited to 9 per cent and the tricalcium silicate to 35 per cent. Similarly, a 1450 cement means that the tricalcium aluminate is limited to 14 per cent and the tricalcium silicate to 50 per cent.

Reverting again to our earlier inquiry into the properties of cement, I might point out that our new cement specification, adopted Jan. 2, 1935, providing that "the tricalcium aluminate shall not exceed 9 per cent and the tricalcium silicate shall not exceed 55 per cent" was not approved by the Bureau of Public Roads. As time passed and as interest increased, we finally adopted and had approved our present cement Item 15-C (Aug. 26, 1936). This is the same as Item 15 (above turned down) except that the control is by oxide analysis

rather than theoretical compound. We are now using Item 15-C which, for convenience, I will call our Normal Portland Cement.

Again, in view of the interest in this subject and in view of suggestions made, we followed a course of requiring different types of cement with the same aggregates in the same project. We now have some seventy-five projects scattered throughout the State for experimental purposes, a partical list of which is shown in the appendix to this paper. On each project we designated in advance the locations where different types of cement were to be used with appropriate references to these types. To distinguish these different cements they were designated by numerals, such as 850, 865, 935, 950, 1258, 1439, and 1450, and, in this paper, they will be called "Special Cements." In the table listing a number of projects in different counties, with highway numbers, names, etc., and under the words "Nature of Experiment," you will find a reference to "Special Cements." In this paper I cannot devote sufficient time to make a detailed report as to the behaviour of the varying types of cement in the projects shown in the tabulation. Neither does the tabulation contain all of the projects throughout the State in which we have used special or other blends of cement. They are the earlier ones which we have constructed and we intend to make periodic inspections as time passes and appropriate records concerning such inspections. Several of them will be commented upon later in the paper.

Coincident with our studies of experimental projects, we examined a number of concrete structures of great age which apparently have remained durable over a long period. Among these older structures we are impressed with the service record of one of the so-called "natural cements"—Rosendale. Of course, the strength of natural cements, as we all know, is much less than normal portland cement. On the other hand, some of the structures with natural cement give very clear indications of long life (another measure of durability). To keep our pavement strengths within reasonable limits and, at the same time, to increase their useful life, we have built a number of experimental pavements in which we have incorporated varying percentages of natural cements with normal cement. In most cases we have used one bag of natural and six bags of normal, although in a few cases we have used two bags of natural cement and five bags of normal. This type of experiment, which we refer to as "blended cement" is shown on the tabulation of experimental projects by the word "natural." By the term "blended cement" I do not mean combining them at the point of manufacture but at the mixer. Probably the most noticeable results that we have observed in the field concerning blends with

natural cement and with special cements are on the so-called Marcy Hill project on a road leading northerly from Utica towards Watertown.

MARCY HILL PROJECT

This is a concrete pavement generally three lanes wide, 8 in. uniform depth, and built upon a 6-in. run of bank gravel foundation course. It is of 1:1¾:3½ mix and was completed in the fall of 1936. It extends northerly from Station 170 on the south to Station 395+96 on the north. Going northerly includes a +7 per cent grade 6500 feet long and a −7 per cent grade 3000 long feet. It is in the so-called snow belt of the State and carries about 5000 vehicles in twenty-four hours. It is subject to ice and sleet storms and is treated with calcium and sodium chloride salts many times in the winter months. There is no doubt about that on many occasions the abrasive chloride treatments consist largely of chlorides rather than cinders, sand, etc.

The project contains special cements 935, 950, 1439, 1450, and normal 15-G. Each of the cements was used alone and each was blended with one bag of Rosendale and then with two bags of Rosendale natural cement. There were a number of places where we wished to obtain the benefits of high-early-strength cement—to be opened to traffic in forty-eight hours. We obtained these benefits by increasing the amount of normal cement (15-G) from seven bags per batch to ten bags per normal batch, decreasing the sand in the latter by an amount equivalent to three bags of the added cement. The normal mix was 1:1¾:3½ and a 7-bag batch made about 30 ft. of concrete. To obtain the same volume of concrete—rich mix—we used ten bags of cement and decreased the sand. The rich mix is 48-G.

In an effort to describe the service conditions, showing the presence or absence of scale, I took pictures and prepared a few slides. Unfortunately the weather and pavement conditions were not very favorable for taking pictures.* It does however seem evident that the blended cements have, so far, shown a better service record than the normal cements in the project.

STONE HOUSE-INDIAN CASTLE HIGHWAY, HERKIMER COUNTY

This project is on heavily traveled touring route 5-S (alternate to 5) which descends into the Mohawk Valley about one mile south of the city of Little Falls in Herkimer County. It is a 20-foot pavement, 8:7:8 section, 1:1¾:3½ mix and some 6100 ft. of it is on a 7 per cent grade descending easterly. It was finished in 1935 and on the portion from Station 0+00 (west) to Station 166+75 (east) three different types of cement were used—765, 935, and 950, each with the same sand

*Such of the pictures as might be reproduced to show relative surface conditions are so few, considering loss of detail in reproduction, as hardly to be representative of the results of the project. The pictures are therefore omitted.—EDITOR.

and stone aggregates. Since completion it has gone through four winter seasons and as this project is in a snow-belt country and on a 7 per cent grade, it has been frequently treated with chloride salts to remove ice from the pavement. An examination of the pavement discloses that the concrete in which a 765 type of cement was used is, at this time, free of scale. There are evidences of varying amounts of scale, however, where 935 and 950 cements were used.

ONONDAGA COUNTY ROAD

At the February meeting in 1938 of the Association of Highway Officials of the North Atlantic States, R. B. Traver, County Superintendent of Highways of Onondaga County, presented a paper "Chloride-Salts-Resistant Concrete in Pavements." In this paper Mr. Traver described some accelerated field tests, using calcium and sodium chloride salts for removing ice on pavement. As his experiences are comparable with our own, I asked his permission to include in this paper a few comments on the nature of the experiment and some of the results.

About four years ago, Onondaga County constructed a concrete road leading from the city of Syracuse to Oneida Lake. They placed in certain sections in one of the lanes, a blended-cement concrete, using one bag of Rosendale natural cement and five bags of normal portland cement in a normal six-bag mix. To obtain quick results they concentrated in each treatment the amount of salt that would normally be applied during a whole year of average weather. This was about .65 lb. per sq. yd. The slabs in one lane were made of normal portland cement and in the other lane they used a blend of normal portland and Rosendale natural cement. The chloride treatments were identical on all types of concrete. A test section 150 feet long was set aside for sodium chloride salts and a similar length for calcium chloride salts, the two sections being separated by about 250-foot intervals. After three applications (about equal to three winters of treatment), they made a study of results.

The study was made about a year ago. There is little material change from that date to the present. It might be difficult to measure or estimate the added depth of scale, if any, between the conditions a year ago and the condition of this date.

Normal portland-cement concrete does not develop immunity to chloride attack in three or four years if the attack is in concentration sufficient to melt the ice. Also, I think it evident that the blended-cement concrete is highly resistant to both sodium and calcium salts in both normal and highly concentrated treatments.

We are sufficiently impressed by the favorable behaviour of concrete using a blend of one part Rosendale natural and six parts normal cement as compared with normal cement alone that we now use that blend as standard construction for concrete pavement. We have also used blends of Akron and Louisville natural cement and will probably continue our experiments with them in the field. Up to this time, however, neither of them has shown a behaviour comparable with or consistent with the behaviour of Rosendale. We are endeavoring to identify the property or properties of Rosendale cement that seems to improve the durability of concrete pavement, especially in protecting the pavement from the attacks of calcium and sodium chloride salts used for removing ice. We suspect that one favorable property may be due to a process of manufacture.

To further our investigation and to enable us to coordinate field and laboratory behaviour, we built in 1938 the Northern State Parkway, Suffolk County. In this project we used four different types of cement which I will identify as Normal 15-C, A. S. T. M. Standard, Old Fashioned, and a Normal cement (15-C) to which was added a grinding compound. These cements were used separately and then each was blended with one bag of Rosendale natural cement, resulting in a total of eight types of concrete. Two cores were then taken from each of the eight sections. They were brought to the laboratory and subjected to 25 cycles and to 50 cycles of freezing test, using a 10 per cent solution of calcium chloride. The losses at the end of 25 cycles and of 50 cycles of each pair of cylinders are shown in Table 3.

Believe it or not, the grinding compound used was a so-called beef fat in an amount of about 1/20 per cent. From these data, now incomplete, we can easily get the impression that the behaviour of normal cement, using a grinding compound, is at least comparable with a normal cement blended with one bag of Rosendale natural. These are the last two items in Table 3. The losses, with grinding compound, at the end of 25 cycles were 0.3 per cent and 0.7 per cent respectively, and at 50 cycles the losses were 1.6 per cent and 2.3 per cent respectively. You will also note that these average losses are materially less than the losses on the other cements and blends of cement.

Many of you will hear considerable discussion on the subject of grinding compounds, be they beef fat, calcium stearate, fish oil, and what not. I do not feel competent or prepared to discuss grinding compounds in any great detail at this time but we are satisfied that their use increases water-repellant properties of cement and increases workability of concrete.

TABLE 3

Results of freezing tests—10% Ca.Cl. Northern State Parkway—N.S.P. 38.2—Nassau County. Fine Aggregate—Colonial S. & S. Co. Coarse Aggregate—N. Y. Trap Rock Co. (rock). Cement as shown. Laid July 25 and 26, 1938. Tests began August 22, 1938.

Cement	Original Weight Grams	25 Cycles			50 Cycles		
		Wt.	Loss		Wt.	Loss	
			Grams	%		Grams	%
15C	5230	0@25	5230	100	.....	.....	.....
	5290	3385	1905	36.0	0@39	5290	100
			Av. =	68.0			
15C + R.	5600	5335	265	4.7	4100	1235	23.2
	5795	4985	810	14.0	3495	2300	39.6
			Av. =	9.4		Av. =	31.9
A. S. T. M.	5600	0@25	5600	100	.....	.....	.....
Standard	5570	2960	2610	46.8	0@43	5570	100
			Av. =	73.4			
A. S. T. M. + 1R	5450	3650	1800	33.0	3140	2310	42.4
	5590	3800	1790	32.0	3490	2100	37.6
			Av. =	32.5		Av. =	40.0
Old Fashioned	5330	3550	1780	33.3	0@38	5330	100
	5400	3910	1490	27.6	0@35	5400	100
			Av. =	30.5			
O. F. + 1R	5330	4385	945	17.7	4220	1110	20.9
	5560	4500	1060	19.1	4200	1360	24.5
			Av. =	18.4		Av. =	22.7
With Grinding	5220	5210	10	0.2	5170	50	1.0
Compound	5450	5430	20	0.4	5330	120	2.2
			Av. =	0.3		Av. =	1.6
With Grinding	5200	5170	30	0.6	5055	145	2.8
Compound	5140	5100	40	0.8	5050	90	1.8
+ 1R			Av. =	0.7		Av. =	2.3

In conclusion, permit me to say that this is the first biography of cement concrete pavement I have ever attempted to assemble. Permit me also to say that I strongly suspect that it will be my last attempt to condense such a broad subject.

In an Appendix* I have listed 21 projects so that you may locate them in one of the ten districts of the state. If you are further interested in examining them at any time I know that any of our district engineers will be glad to show them to you in the field (and perhaps others) of which we have about 75 scattered throughout the state.

Discussion, to close in the September Supplement, should reach the A. C. I. Secretary by July 20.

*Page 578.

APPENDIX

For information, I have listed the following experimental projects constructed in various parts of the state beginning with the year 1925. Time does not permit a detailed report on each, but the objectives sought or comparisons made are shown in the last column, and a few of the outstanding ones were discussed in this paper.

County	S.H. No.	R.C. No.	Length In Miles	Highway Name	Year Built	Nature of Experiment
Erie	525	1426	2.98	Lawtons-Gowanda	1925–5	Comparing slag, stone and gravel coarse aggregate
Chautauqua	8304		7.65	Dunkirk-Silver Creek	1928–9	Two-course pavement compared with monolithic. Also various cements
Genesee	1916		3.77	Oakfield-Elba-Byron, Pt. 1	1931	Imported aggregates (L. I. sand and Trap Rock) compared with local
Erie	523	1999	7.60	Hamburg-N. Collins	1931	Two-course pavement compared with monolithic
	524		3.31	N. Collins-Lawton		
Cattaraugus	5258	2165	0.89	Little Valley-Randolph, Pt. 3	1931–2	Seaboard bank sand from L. I. compared with local materials.
	5258A	3329	4.70			
Genesee	8089			Batavia-Bergen, Pt. 2	1934	Subgrade Felt
Herkimer	8417	3572	3.15	Stone House-Indian Castle	1934–5	Natural & Special cements.
	831		4.49	Little Falls-Montgomery Co. Line		
Albany	176	3590	1.73	Albany-Schenectady, Pt. 1	1934–3	Natural & Special cements.
Albany	385		5.13	Albany-Schenectady, Pt. 2		
Schectady	179		0.72	Schenectady Turnpike		
Otsego	708	3591	0.11	Sidney-Unadilla	1934–3	Special cements.
Otsego	5202		1.53	Unadilla Village		
Otsego	5229		3.23	Unadilla-Wells Bridge		
Delaware	8254		0.11	Unadilla-Franklin, Pt. 1		
Erie	584			Skinnersville-New Home Bridge		
	89	2293	0.31	Transit, Pt. 2		
	1949	....	6.53	Buffalo-Millersport, Pt. 2	1933	Monolithic Slag. Various methods of curing
Greene	5685	3639	0.09	Catskill Village	1935–7	Natural and Special cement.
Greene	1934		0.12	Catskill-W. Coxsackie, Pt. 1-A		
Greene	8414		0.32	Catskill-W. Coxsackie, Pt. 1-B		
Greene	5198		4.35	Coxsackie-Ravena, Pt. 1		
Greene	5370		2.51	Coxsackie-Ravena, Pt. 2		
Albany	5652		0.49	Ravena-Greene Co. Line		
Oneida	933	3649	4.26	Deerfield Cor.'s.—Marcy Hill	1935–6	Natural and Special Cement.
Oneida	931		2.06	Marcy Hill—Trenton		
St. Lawrence	5283	3643	8.53	Gouverneur-DeKalb	1935–6	Natural and Special cement.
St. Lawrence	9236		3.06	Richville-DeKalb Jct.		
Jefferson	5304	3642	6.41	Clayton-Alexandria Bay	1935–6	Natural and Special cement.
Jefferson	182		0.07	Redwood-Alexandria Bay, Pt. 1		
Jefferson	1130		0.31	Alexandria Bay: Church Street		
Seneca	8452	3620	5.16	Sheldrake-Taughannock Falls, Pt. 1	1935–6	Natural and Special cement.
Seneca	8436		5.04	Sheldrake-East Varick		
Livingston	8490	3746	5.57	Alexander-Canawaugus, Pt. 6	1936–7	Natural and Special cement.
Livingston	5273		0.54	Caledonia-Avon		
Tompkins	5214	3808	0.29	Ithaca-Newfield	1936–7	Natural and Special cement.
Tompkins	454		1.99	Ithaca Road		
Dutchess	8492	....	3.20	Rhinebeck-Hudson, Pt. 1	1936–7	Natural and Special cement.

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Discussion of a paper by E. C. Lawton:

Durability of Concrete Pavements—Experiences in New York State*

BY FRED HUBBARD†

Both Mr. Lawton and the New York Division of Highways are to be complimented on the fact that experimental projects built in the State of New York are not lost sight of after the initially contemplated data have been obtained but are systematically examined in years following. We wish to emphasize the value of comparative service records of engineering structures when all the facts surrounding their construction are known. Such compilation of facts is truly a contribution for the advancement of practical engineering knowledge.

In the course of studies made several years ago by the National Slag Association, experimental data bearing on the same subject were developed. A number of fine and coarse aggregates from several western New York sources were tested to determine their resistance to sodium sulfate and freezing and thawing soundness tests. At the same time, mortars and concretes containing these aggregates were made and tested for durability as measured by their resistance to freezing and thawing.

The aggregates tested included Blast Furnace Slag from four sources, gravel from three sources, crushed stone from two sources, and sand from seven sources.

The coarse aggregates for use in concrete were graded uniformly from 1½ in. to No. 4 sieve, identical gradings being used in every instance. The sands were used in the gradings as produced and were, for the most part, rather coarse, varying in fineness modulus from 3.02 to 3.53.

*JOURNAL, Amer. Concrete Inst., June 1939; *Proceedings* Vol. 35, p. 561.
†Director of Research, National Slag Association.

TABLE 1—RESULTS OF COMPRESSIVE AND FLEXURAL STRENGTH TESTS ON CONCRETE COMPOSED OF SAND B AND VARIOUS COARSE AGGREGATES

Coarse Aggregate	Compressive Strength—Lbs. per Sq. In.					Flexural Strength—Lbs. per Sq. In.				
	7 da.	14 da.	28 da.	90 da.	Avg. Rel. Val.	7 da.	14 da.	28 da.	90 da.	Avg. Rel. Val.
A (Slag)	4345	5352	6077	6899	107	754	867	930	971	108
B "	4492	5158	5616	6684	104	738	809	834	938	102
C "	4371	5069	6014	6551	104	704	763	821	825	96
D (Stone)	4322	5157	5636	6555	103	713	825	917	929	104
E "	4209	4820	5519	6479	100	754	821	879	925	104
F (Slag)	3930	4564	5190	5900	93	742	754	825	967	101
G (Gravel)	4334	4640	5203	5919	95	667	742	784	825	93
H "	4138	5024	5248	5953	96	617	713	733	809	88
I "	4017	4913	5317	6215	97	742	759	838	1029	104

TABLE 2—RESULTS OF COMPRESSIVE AND FLEXURAL STRENGTH TESTS ON CONCRETE COMPOSED OF SLAG F AND VARIOUS SANDS

Sand	Compressive Strength—Lbs. per Sq. In.					Flexural Strength—Lbs. per Sq. In.				
	7 da.	14 da.	28 da.	90 da.	Avg. Rel. Val.	7 da.	14 da.	28 da.	90 da.	Avg. Rel. Val.
A	3625	4193	4659	5517	96	646	742	754	834	96
B	3930	4564	5190	5900	104	742	754	825	967	106
C	3858	4437	5084	5766	102	667	704	754	829	95
D	3948	4572	5227	5918	105	713	784	842	883	104
E	3473	4000	4472	5048	90	638	680	717	809	92
F	4273	4742	5134	6036	107	729	828	892	955	110
G	3590	4323	4622	5441	95	638	696	775	825	95

TABLE 3—RESULTS OF COMPRESSIVE AND FLEXURAL STRENGTH TESTS ON CONCRETE COMPOSED OF GRAVEL G AND VARIOUS SANDS

Sand	Compressive Strength—Lbs. per Sq. In.					Flexural Strength—Lbs. per Sq. In.				
	7 da.	14 da.	28 da.	90 da.	Avg. Rel. Val.	7 da.	14 da.	28 da.	90 da.	Avg. Rel. Val.
A	4453	5056	5576	6312	104	680	775	783	888	96
B	4334	4640	5203	5919	98	667	742	784	825	93
C	4213	4899	5530	6322	102	675	759	805	904	97
D	4152	5056	5451	5921	100	754	850	900	967	107
E	3945	4683	5199	5717	95	704	746	809	879	97
F	4593	5312	5840	6745	109	783	800	967	1067	112
G	3852	4469	4688	5684	91	709	738	834	875	98

TABLE 4—RESULTS OF COMPRESSIVE AND FLEXURAL STRENGTH TESTS ON CONCRETE COMPOSED OF STONE D AND VARIOUS SANDS

Sand	Compressive Str.—Lbs. per Sq. In.					Avg. Rel. Val. Slag, Stone, Gravel	Flexural Str.—Lbs. per Sq. In.					Rel. Val. Av. Slag, Stone, Gravel
	7 da.	14 da.	28 da.	90 da.	Av. Rel. Val.		7 da.	14 da.	28 da.	90 da.	Av. Rel. Val.	
A	4594	5054	5425	6370	104	101	646	788	863	950	96	96
B	4322	5157	5636	6555	105	102	713	825	917	929	100	100
C	4209	5088	5450	6167	101	102	679	767	850	896	95	96
D	4213	5049	5754	6138	102	102	792	829	950	1092	109	107
E	3875	4558	4828	5399	91	92	684	754	817	867	93	94
F	4379	5359	5587	6668	106	107	792	863	921	1088	108	110
G	3816	4340	5083	5614	91	92	759	788	842	950	99	97

TABLE 5—SUMMARY OF THE RESULTS OF SOUNDNESS TESTS ON COARSE AGGREGATES, AND CONCRETE

Coarse Aggregate	Sodium Sulphate Loss in % 4000 g 1½"–¾" 2000 g ¾"–⅜" 1000 g ⅜"–No. 4 Weighted Average	Freezing and Thawing Loss in % 25 Pieces 2" Aggregate	Freezing and Thawing Loss in % Concrete Sand B
	5 Cycles	*50 Alternations*	
A (Slag)	0.0	0.0	0.0
B "	0.2	0.0	0.4
C "	0.7	0.0	0.0
D (Stone)	1.4	2.2	0.0
E "	1.2	1.4	0.0
F (Slag)	6.1	5.7	0.8
G (Gravel)	3.1	1.0	1.3
H "	7.1	11.0	0.0
I "	13.0	7.0	0.0
	10 Cycles	*100 Alternations*	
A (Slag)	0.2	1.4	1.1
B "	0.3	0.0	0.9
C "	0.8	0.0	0.6
D (Stone)	3.1	5.2	0.5
E "	4.0	4.8	0.1
F (Slag)	7.9	7.2	1.7
G (Gravel)	8.3	1.5	1.6
H "	13.0	21.9	0.5
I "	20.5	10.4	0.0
	15 Cycles	*150 Alternations*	
A (Slag)	0.4	2.9	1.5
B "	0.4	1.9	1.4
C "	1.0	0.0	0.7
D (Stone)	3.7	12.2	9.4
E "	6.1	6.6	1.0
F (Slag)	9.6	13.3	2.8
G (Gravel)	15.1	5.1	1.6
H "	17.2	29.6	0.6
I "	26.1	14.2	0.0
	20 Cycles	*200 Alternations*	
A (Slag)	0.8	2.9	2.3
B "	0.6	1.9	2.1
C "	1.1	0.0	1.1
D (Stone)	4.6	17.7	100.0
E "	7.7	7.6	17.1
F (Slag)	9.9	14.8	3.8
G (Gravel)	25.0	10.8	1.8
H "	22.3	39.7	100.0
I "	31.4	20.3	0.4

Concrete mixes using normal portland cement were designed according to the fineness modulus theory for a cement factor of 6.5 sacks per cubic yard. The sand ratio by dry-rodded volume was approximately 45 per cent for the crushed aggregates and 39 per cent for gravel. The concrete in all mixes was made to the same consistency as represented by approximately 2¼ in. slump and a flow of 175. The resulting net w/c was approximately .76, or 5.7 gal. per sack for the crushed aggregates and .71 or 5.5 gal. per sack for the gravels.

Results of strength tests of the concrete are shown in Tables 1, 2, 3 and 4. Table 1 shows the strengths developed by concrete composed of each coarse aggregate used with sand B. Tables 2, 3 and 4 show the strengths developed by concrete composed of each sand used with Slag F, Gravel G and Stone D, respectively.

TABLE 6—SUMMARY OF THE RESULTS OF SOUNDNESS TESTS ON FINE AGGREGATES MORTAR AND CONCRETE

Fine Aggregate	Sodium Sulphate Loss in %	Freezing and Thawing Loss in %		
	100 g Separate Sizes Weighted Average	100 g Sep. Sizes Wghtd. Average	Mortar Cubes	Concrete Gravel G
	5 Cycles		*50 Alternations*	
A	0.3	0.7	1.3	0.0
B	5.9	5.3	0.0	1.3
C	5.5	10.6	3.2	0.0
D	8.0	4.8	0.9	0.0
E	6.3	4.2	0.0	0.0
F	6.6	5.4	0.0	0.0
G	6.3	5.9	0.0	0.0
	10 Cycles		*100 Alternations*	
A	1.4	1.0	2.1	0.0
B	14.3	7.1	0.0	1.6
C	12.0	13.5	6.8	0.0
D	15.8	7.6	1.6	0.0
E	15.1	6.3	0.0	0.0
F	21.3	7.3	0.8	0.0
G	15.9	7.7	0.0	0.0
	15 Cycles		*150 Alternations*	
A	3.2		7.2	0.4
B	17.9		2.0	1.6
C	19.4		15.5	0.0
D	17.5		4.8	3.8
E	24.2		0.5	0.2
F	23.1		1.4	0.0
G	25.8		0.0	0.0
	20 Cycles		*200 Alternations*	
A	4.0		9.6	1.7
B	21.4		2.8	1.8
C	23.0		20.6	12.7
D	20.8		8.4	37.0
E	29.1		0.7	1.2
F	26.0		3.2	0.5
G	29.5		1.5	0.0

Both the compressive and flexural strengths were very satisfactory, indicating a high quality concrete from the standpoint of strength.

Tables 5 and 6 show the results of sodium sulfate soundness tests on the aggregates and also freezing and thawing tests on the aggregates, mortar and concrete. The mortars were mixed with a water-cement ratio of 1.00 and a flow of 200 ± 5.

It will be noted that all concrete withstood fairly well 150 alternations of freezing and thawing but at 200 alternations several specimens were entirely disintegrated.

The soundness of the coarse aggregates as indicated by the sodium sulfate tests is approximately the same as that indicated by the freezing and thawing tests. The soundness of the coarse aggregates is not in all cases indicative of the durability of the resulting concretes as measured by freezing and thawing. For instance, Gravel G shows considerable loss in the soundness tests but the resulting concrete shows very little failure at the end of 200 alternations of freezing

and thawing. Stone D is a fairly sound material but the resulting concrete shows 100 per cent failure at 200 alternations of freezing and thawing. On the other hand, Gravel H is a very unsound material as measured by the sodium sulfate test and the resulting concrete shows 100 per cent failure at 200 alternations of freezing and thawing.

Slags, as a class, are the soundest of the coarse aggregates as indicated by both sodium sulfate and freezing and thawing tests. The slag concretes also in all instances withstand 200 alternations of freezing and thawing with very little loss whereas concretes made from one gravel and one stone were entirely disintegrated at 200 alternations.

The sodium sulfate and the freezing and thawing tests give somewhat similar results on the fine aggregates, but neither accurately predicts the durability of the corresponding concretes as indicated by freezing and thawing tests.

The relative soundness of mortar cubes is generally similar to the soundness of the resulting concrete as indicated by freezing and thawing.

The compressive and flexural strengths of concrete are not materially affected by the aggregates, the crushed coarse aggregates (slag and stone), although containing a weaker cement paste, giving slightly higher strengths than the gravels.

The relative strengths of concrete developed by the use of various sands were generally similar when used with each type of coarse aggregate.

Some of the materials used in the Lawtons—Gowanda experimental road are represented in these laboratory studies, the results of which seem to confirm Mr. Lawton's excellent report of the actual service record of the materials.

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Job Problems and Practice

Some will ask questions — Some will answer them — Some will do both

A. C. I. members are invited to use this new JPP department as an informal means toward mutual help. When a problem holds possibilities of general interest the discussion will be briefed in these pages.

If you have a problem—present it; a question, ask it. If you know the answer to a question asked, or if you can contribute something which may help in the solution, or have reason to disagree with the answers or suggestions published, your contribution will be welcome.

If you know of an interesting problem whose solution has already smoothed the way of someone in the field, tell us about it—some other A. C. I. member may need just that information.

The "answers" are the answers of individuals to whom the questions are referred and not of the A. C. I. as an organization.—EDITOR

Safe length of haul for ready-mixed concrete?

(No. 35-35 Apr. '39 Jl. p. 421)

Q—Apropos of the article by Stanton Walker, I would be interested in further information on this subject.

A year ago last summer I specified transporting ready-mixed concrete some 50 miles from New Haven to Hadlyme, Connecticut. This was due to the fact that good local aggregates were not available at Hadlyme and, as the job was a swimming pool requiring high-grade concrete, I wished to have the proper mix prepared at a central mixing plant here in New Haven. I specified a six-gallon mix with enough cement for good workability and laid out a very careful schedule planning to pour the concrete within two hours after it was wet mixed at the plant (no facilities for dry mix). The material was delivered in transit mixers.

Owing to delay on the part of the contractor, the forms were not ready to receive the concrete upon arrival and, as it was not possible to intercept the trucks upon the road, and one delay led to another, we eventually had three drums containing three and one half yards of concrete each, which had to be mixed between five and six hours before it was possible to place it.

By adding a little water at a time, we succeeded in keeping the drums rolling and eventually placed the whole batch which appeared to have the consistency corresponding to a two or three inch slump (no test made as we had no slump cones on the job). It was certainly very plastic and went in beautifully. As far as I know, this job was entirely successful and I am wondering if other engineers or builders have had similar experiences.

A—It is entirely within reason to have obtained the results described, although I suspect that there was an element of luck involving a favorable combination of circumstances; that the temperatures were not particularly high and that the cement happened to be one having characteristics adapted to the condition.

My discussion of this subject in the April JOURNAL referred to tests by Miles N. Clair. In his report results are given for periods of agitation up to 3.58 hours for an open top body and to 3.0 hours for a revolving drum body. The slump was reduced from about 7½ in. to 2 in. for these extreme periods of agitation, but the strength was relatively little affected.

The "Tests of Retempered Concrete" by Gonnerman and Woodworth referred to in my recent discussion covers tests for concrete which had been allowed to stand up to 6 hours. Two of the authors' conclusions are the following:

1. The most striking result brought out by this investigation was the small loss in compressive strength due to standing for periods up to 6 hr. (protected from evaporation) of concrete remixed without the addition of water. When the mixtures remained plastic and workable the loss was practically nil. After the concrete ceased to be plastic the strength fell off rapidly. There was always a reduction in flow with an increase in standing period.

2. In the concrete remixed with the addition of water to restore original flow after standing for periods up to 6 hr. (protected from evaporation) the strength was reduced in accordance with the increase in the water-cement ratio resulting from the added water.

These data seem pertinent to the subject under discussion. There would, of course, be little or no evaporation of water from a closed drum transit mixer.

The New Haven experience and the preceding data should not be used to encourage permitting excessive periods between mixing and

placing concrete. The data show that long periods may be permitted, under certain circumstances, without harm. On the other hand, experience has shown that combinations of circumstances may make the use of excessive periods dangerous to the quality of the concrete.—STANTON WALKER

Safe rate of cooling for concrete? (No. 35-39)

Q—What is considered a safe and practical rate of cooling for a relatively thin section of concrete after being cured for five days at temperatures between 50° F. and 70° F. with the ambient air temperatures between 20° F. and 40° F.? The concrete contains 6 bags of "moderate heat of hardening" portland cement per cubic yard of concrete, with 5.5 gallons of water per bag of cement, giving an average slump of 3½ in. The concrete arch sections are 18 in. thick and 30 ft. long, each section containing about 70 cu. yds. of concrete. This question is in connection with the construction reinforced concrete pressure conduit, 16 ft. inside diameter.

A—After concrete has been cured at 50 to 70° F. for five days, it should be proof against freezing and only needs be cooled slowly enough so that it will not crack. The most satisfactory manner of insuring gradual cooling is usually provided by leaving the wooden forms in place for several days after the artificial heat is removed. There is little danger of cooling too rapidly as long as wooden forms are in place. A guide to the proper cooling might be obtained by observing the temperature change of the concrete immediately under the forms. Many factors are controlling, but a sudden cooling of the concrete itself of more than 20° F. overnight would be likely to cause cracking.—P.R.C.

A—With a cement content of 6 bags of moderate heat cement per cubic yard of concrete the adiabatic temperature rise due to heat of hydration for 5 days is about 75 degrees. Nevertheless, with such a thin section the average temperature rise would not exceed 20 degrees, which would occur within the first day, and by the end of the 5-day curing period the mean temperature would be within a few degrees of the curing temperature. By the end of this time the temperature gradient throughout the section also would not exceed a few degrees—say center temperatures not over 4 degrees above surface temperatures. At the end of the 5-day curing period, therefore, the section may be considered to be at a uniform temperature of, say 60° F.

If both sides of the 18-inch slab are now suddenly subjected to an exposure temperature of from 20° F. to 40° F., say 30° F., the maxi-

mum stress conditions would occur at the end of about one hour of exposure, at which time the center temperature would just be starting to lower and the full 30-degree gradient would exist throughout half the thickness of the section. A 30-degree temperature gradient under such conditions might cause cracks. It would take about 5 hours to lower the center temperature to less than half the initial temperature difference.

This suggests lowering the temperature in two steps; a first step of say 20 degrees at the end of the 5-day curing period, and the second step of 10 degrees about 5 hours later. This would subject the concrete to no greater than a 20-degree temperature differential at any time, which is considered to be safe.

If forms are left on during this time, the length of exposure to the first step must necessarily be longer than 5 hours and can best be determined by observing the temperature of the interior and making the second step at such time when the interior temperature has lowered 10 degrees.—CLARENCE RAWHOUSER

Ready-mixed concrete's increased slump in winter haul (No. 35-40)

Q—In the delivery of warm ready-mixed concrete from central mixing plants, instances have been noted where the slump increased during time of delivery. This characteristic has been observed as the reverse of the performance with the same aggregates and cement in summer operation, when the concrete loses slump and becomes stiffer after a half or three-quarter hour haul than when freshly mixed. The increase in slump with winter deliveries is probably not universal, but is observed from time to time. Why should any concrete increase in slump during a winter haul of say one-half hour?*

How to design concrete mixtures?

(No. 35-21 Jan. '39 Jl. p. 211)

Mr. Stanton Walker has described a simple procedure for designing concrete mixes, which he offers for the benefit of engineers who do not wish to go into the more complicated systems. With certain additions, the method is substantially the one we have used almost exclusively for several years in our laboratory investigations, and we believe Mr. Walker's modesty has led him to underemphasize the value of this method. We are convinced that it is dependable and, in experienced hands, capable of giving very close to the optimum combination of given materials with the least effort.

*See also No. 35-35 April '39 Jl., p. 421, and this issue p. 581.—EDITOR

One may wonder why this excellent method has never been published before. One reason may be that those who have used it, like ourselves, have built up the procedure gradually to meet their own requirements, but feel that it might not be accepted by engineers because it lacks complete mechanical synthesis. Another reason may be that a close approach to the optimum design requires judgment and experience on the part of the operator, and the method therefore depends upon a combination of art and science. But there is no certain way, regardless of the method of design, to compute the water required for a given mix without trial batches. If the necessity for trial batches is admitted, no extra labor is caused by the substitution of study of the characteristics of the trial mixtures for the usual computations. As one gains experience, his preliminary trials come closer to the mark, and indicated changes are made with greater certainty.

For the encouragement of further discussion, I am outlining herewith the scheme we use for selecting aggregate proportions. I may say, however, that our problem is not usually one of furnishing guaranteed strengths, but rather of making the best of the materials submitted in trying to produce a satisfactory and economical concrete for the purpose in view.

Our procedure consists in first making and plotting on a single sheet of cross section paper the sieve analysis of the fine and coarse aggregates. Then a so-called ideal gradation curve is drawn in according to the Furnas-Anderegg method, in which each sieve fraction bears a definite ratio to the next smaller one. The ratio may be 1.25 for rounded gravel or it may be 1.15 for crushed material; the selection is based on aggregate shape. This curve is drawn to the same maximum size as that of the aggregate submitted, but we use it as a guide rather than as a result to be obtained as nearly as possible, and we depart from it in the following manner. It is our custom to plot gradations on a semi-log plot—cumulative per cent passing the sieves of the series 100, 50, 30, etc., vs. the sizes of the sieve openings. On such a plot the Furnas-Anderegg gradation is a smooth curve with constantly increasing slope from the lower left hand corner to the upper right. From the separate curves of the aggregates in question we try to combine them in such manner that the coarse aggregate portion of the combined curve will lie to the right of the Furnas-Anderegg curve, and the fine aggregate portion to the left. This is not always possible, of course, but in general we prefer a combination that gives more coarse and more fine particles, and fewer of the intermediate particles, than the grading represented by the smooth curve. As a matter of fact we prefer an approach to a gap grading, and if there tends to be

a "hump" on the curve in the neighborhood of the No. 4 sieve size, we recommend scalping the fine aggregate to eliminate it.

Some experience is necessary to make a good stab at the proper proportions of fine and coarse aggregate on paper, for the selection will depend to some extent on cement factor and method of placing, as well as on characteristics of the aggregate itself. But after some familiarity with the procedure, one or two trial batches are usually all that are required to secure a mix that is near the optimum for the conditions imposed. The trial batches are computed and compounded exactly by the procedure and in the sequence indicated in Mr. Walker's communication, the endeavor in the first trial being to use the amount of water necessary to give the desired consistency, even though this may differ somewhat from the computed amount. In this manner the balance between mortar and coarse aggregate can be better judged, leaving the correction of the water or paste content to be made in the second or later trials.

The unit weight of the concrete of each batch is determined by filling a measure very carefully. We over-fill the measure very slightly, smooth off with a flat trowel, and then slide a glass plate across the top with a sawing motion to remove the small excess. From the unit weight and the known weight of the batch, we compute the volume of the batch, the number of batches per cubic yard, and finally the quantity of each material per cubic yard.

In this manner one feels his way rather quickly to the desired end, not entirely by prescribed steps, but partly by his ability to judge the quality of a concrete mixture by handling it and observing it critically. One readily learns to distinguish between the effects of harshness or poor grading and of unbalanced proportions, to determine whether mortar or coarse aggregate is in excess, and to make such adjustments as will give the desired workability with minimum cement content. With this ability he can "design" a mixture out of almost any combination of materials without knowing anything about fineness modulus, voids, or unit weights of the aggregates. On the other hand, knowledge of the specific gravities and absorption of the aggregates is essential for close control of water-cement ratios and quantities of ingredients.—J. C. PEARSON

Mr. Walker's letter seems to the writer a valuable contribution to practical control of concrete mix design. Inasmuch as the writer has followed pretty much the same method of approach as is outlined by Mr. Walker, it is thought that the following data obtained thereby might be of interest.

For concrete in which natural sand and gravel were used as fine and coarse aggregate, virtually the same results have been obtained as those outlined in Tables 1, 2 and 3 of Mr. Walkers letter. For concrete in which crushed limestone was used as coarse aggregate, and a blend of 80 per cent river sand and 80 per cent crushed limsetone sand used as fine aggregate, the results set forth in Table 4 were obtained. The mobility of the concrete for these data was held to a slump of 2½ in.—JOHN C. SPRAGUE.*

TABLE 4

	Maximum size, and fineness modulus of concrete aggregate*					
F. M.	1-in. 5.34	2-in. 6.01	3-in. 6.50	4-in. 6.91	6-in. 7.39	9-in. 7.96
Typical ratio of fine to total aggregate (for W/C = 0.80)	43	38	34	32	28	27
Gal. of water per cu. yd. of concrete (varied only with max. size)	40	34	30	29	25	23
Cement content—bags per cu. yd.:						
W/C = 0.70; Comp. str. = 6500 p.s.i.	7.60	6.45	5.70	5.50	4.75	4.35
W/C = 0.80; Comp. str. = 5600 p.s.i.	6.65	5.65	5.00	4.80	4.15	3.90
W/C = 0.90; Comp. str. = 4800 p.s.i.	5.90	5.05	4.45	4.30	3.70	3.40
W/C = 1.00; Comp. str. = 4000 p.s.i.	5.30	4.55	4.00	3.85	3.35	3.05
W/C = 1.10; Comp. str. = 3400 p.s.i.	4.85	4.10	3.65	3.50	3.05	2.80
W/C = 1.20; Comp. str. = 2800 p.s.i.	4.45	3.80	3.35	3.20	2.80	2.55
W/C = 1.30; Comp. str. = 2200 p.s.i.	4.10	3.50	3.10	3.00	2.55	2.35
Unit weight of concrete in lbs. per cu. ft.— W/C = 1.00	151	153	155	156	157	158
Comp. str. per bbl. cement per cu. yd. of Concrete—p.s.i.						
W/C = 0.70	3400	3900	4350			
W/C = 0.80	3300	3800	4300	4700	5250	5700
W/C = 1.00	3150	3600	4000	4300	4800	5150
W/C = 1.10	2850	3300	3700	4000	4450	4750
Aug. Cement efficiency (6" = 100%)	64	74	83	90	100	107

*Fine aggregate has a F. M. of 3.25, consisting of 80% river sand and 20% limestone sand; coarse aggregate consists of crushed limestone.

Equipment for washing down concrete for bonding new concrete? (No. 35-41)

Q—I should like information as to equipment for use in washing down concrete at pressures of 300 to 350 lb. per sq. in., preparatory to bonding new concrete and similar information as to equipment for sandblasting.

A—I know of no equipment designed to permit pressures as high as those you indicate. On all of the construction jobs with which I am familiar the air-pressure ranges from 90 to 100 lb. per sq. in. and the water pressure may be 40 or 50 lb. per sq. in. The nozzle for the air-water jet is a very simple device which the contractors make out of ordinary steel pipe. A sketch of such a nozzle is shown (Fig. 1). The diameter of pipe to be used will naturally depend upon the magnitude of the work of clean-up. On most dam construction the pipe used is

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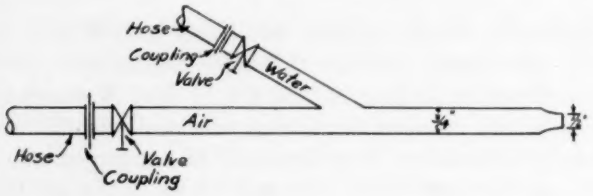

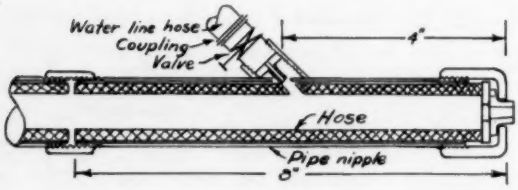

FIG. 1—(TOP); FIG. 2—(BOTTOM)

5/8 to 3/4 in. For small areas such as have been included in laboratory investigations, the diameter of the pipe has been 3/8 in.

For wet sand-blasting on the medium size small job there can be used ordinary dry sand-blasting equipment with a modified form of nozzle having a water inlet. (Fig. 2).

For wet sand-blasting of concrete we have found that a short orifice such as that shown in the sketch is much better than the long tapering orifice commonly used in dry sand-blasting operations. In the operation, only enough water is used to keep the area in front of the jet washed clean of sand.

The earlier method of clean-up (with the air-water jet about the time of final set, followed by a later clean-up just before placing the next left of concrete) has now been abandoned by the U. S. Bureau of Reclamation, and in its place this Bureau is now using a single sand-blast clean-up applied just before placing the next left. Results of field and laboratory investigations indicate that this method of clean-up is very effective in securing high bond strength. Deep cutting to expose pieces of coarse aggregate appears to be unnecessary. In fact, best results seem to be obtained where the degree of cutting is merely sufficient to cut away the surface film of cement paste, exposing the particles of fine aggregate.*

Concrete durability and the w-c ratio (No. 35-42)

Yesterday a young engineer said to me: "Our investigations of concrete durability by means of freezing and thawing tests have shown markedly greater resistance for the lower water-cement ratios.

*The reader is referred to discussion, p. 188 - 1, this JOURNAL of the paper (Jan. 1939 Jl.) by Charles E. Wuerpel, "Tests of Potential Durability of Horizontal Construction Joints."—EDITOR

Consequently rich mixes have become common practice, often as much cement as 7 sacks per cu. yd. being used. This, however, has resulted in considerable checking and cracking. What is the answer?''

My answer was somewhat as follows: Although it is true that durability is related to the water-cement ratio, a point commonly overlooked is that, for a given cement content, it is more closely related to the unit water content of the concrete. Thus, there is still better assurance of durability if the reduction in water-cement ratio may be accomplished primarily by reduction in water content and only secondarily by increase in cement content. Of course an essential degree of workability must be retained to allow good workmanship in placing the concrete, but often mixes in use, ordinarily reduced in water-cement ratio merely by adding cement, could well be revised in one or more of the following particulars, without loss of the essential degree of workability, and result in a reduced water-cement ratio by means of a reduced water content:

(1) Regrading to a larger maximum size which requires less mortar for equivalent workability.

(2) Reduction of the commonly excessive percentage of fine aggregate.

(3) Improve the grading of fine aggregate. The water requirement of harsh sands may be reduced by blending with a small amount of suitable fine sand.

(4) Make every improvement possible in handling and placing equipment and use vibrators so that the stiffest practicable consistency may be successfully handled and placed.

Although test specimens may show greatly superior durability for concretes in which the water cement ratio is reduced merely by addition of cement, the fact that structures of mixes modified on this basis show considerably more checking and cracking, raises the doubt about the benefits apparent in the small unchecked test specimens. These may be largely or more than offset by the weathering forces of disintegration, to bring about equally early destruction of the high-cement-content, low-water-cement-ratio concrete.

The checking and cracking of such concrete is due to the higher internal temperatures reached during the hardening period which results in abnormal thermal stresses. Thus, where the water-cement ratio may be reduced only by adding cement, serious consideration should be given to the selection of a cement having the lowest heat of hydration which is otherwise suitable for the work, and precautions should be taken to place the concrete at the lowest temperature possi-

ble over 45° or 50° F., and to keep it as cool as possible above these temperatures during an ample period of thorough curing.—LEWIS H. TUTHILL.

Bentonite cement grouts (No. 35-43)

A problem was suggested by the contractor who had several hundred yards of grout to pump into a restricted area. He was using a neat grout, with a w/c of .60 by volume which required about 25 sacks of cement per cu. yd. By using a 2 per cent mixture of bentonite and water and adding to this calcium chloride equivalent to 4½ per cent of the cement used, a grout mix with double the water content and with about the same flowability and shrinkage before initial set using only 16 sacks of cement per cu. yd. was obtained. This grout developed a 28-day strength of about 2000 lb. which was adequate and saved the contractor about $3.00 or more per cu. yd.

Another successful use was made of bentonite at the time of a serious leak in the cofferdam which developed a flow of 20 to 30,000 gal. per minute, over 50 feet below the river level. A 2 per cent to 5 per cent solution of bentonite was used to mix with cement, to make a grout into which sawdust, shavings, and other materials were mixed. This was pumped into the water channel. Bentonite resists the erosive action of flowing water and this allowed the cement to set up before it could be washed away.—O. G. PATCH.

Hot lime in ready-mixed concrete (No. 35-44)

Q—We went into the ready mixed mortar business a year or so ago as many ready-mixed concrete manufacturers have done and our plant has been pushed to capacity. On that account, we have not, on all occasions, properly aged the putty we have sold and on one or two occasions we have incorporated "hot" lime putty into ready-mixed concrete. In other words, we would be interested to know what effect, if any, the addition of freshly manufactured calcium hydroxide, in mixed concrete would have on the portland cement contained in the mix, while the concrete is still plastic or after the concrete product has aged.

A—There are two general types of limes—high calcium and dolomitic. Each of these is furnished to the trade as a dry hydrate and as a quick lime. A good grade of high calcium quick lime, particularly in the pulverized state, will slake very radiply and vigorously with the evolution of considerable heat. If this slaking action is allowed to go on until the temperature of the mass is 150° Fahrenheit or below, there should be no danger of the existence of unhydrated parti-

cles, and there should be no burning of the particles if the slaking is correctly done. The addition of such a putty at temperatures below 150° F. to a mortar or concrete will not be detrimental to the structure as far as the basic materials themselves are concerned. On the other hand a dolomitic quick lime when slaked in the same way may have a real proportion of its particles in the unslaked condition. In mortars where a high proportion of such a lime exists, and the putty is used hot, continued hydration may often produce some detrimental results. If this dolomitic quick lime is slaked in large volume so that the heat of hydration aids in the speed of hydration a very large proportion of the magnesia is hydrated. In this instance the temperature of the entire mass may still be from 80° to 100° F. The time of slaking, however, usually runs from a minimum of seven or eight hours, under proper conditions, to 72 hours. The usual dolomitic hydrate which is speedily slaked and then dried may often have a large proportion of its magnesia still unhydrated. If such a product is soaked before using additional hydration takes place. If, however, it is added in the dry state and immediately incorporated into mortar this hydration may take place in the structure. Again, if the lime content of a mortar is high, under these conditions, some detrimental effect may be expected.

The small additions of either lime to concrete which are now being advocated, and with proper distribution of lime particles by thorough mixing, should not produce any serious results in actual practice.

It is very desirable to sound a warning to any one contemplating the use of hot lime. The danger of unslaked particles getting into a structure in quantity is a real one. If, however, the manufacturer and the contractor really know how to handle lime and really desire to handle it correctly, I believe no troubles will be experienced. I saw this sort of thing being done in Chicago and after ten minutes had elapsed during a slaking mix I was able to put my hand in the mass without experiencing any great discomfort. This would indicate a temperature well below 150° F. Most of this sort of thing is done in the winter as a temperature protection, and we must bear in mind that at such times the loss of heat from the mixture is more rapid.—W. C. Voss.*

A—For the best answer to the question the questioner should answer these three questions: (1)How much time elapses between the addition of the water to the lime and the use of the putty in the concrete mix; (2) What size screen was the milk of lime screened through after

*Professor in charge, Course in Building Engineering and Construction, M I. T., Cambridge.

the first boiling action of the hydration had taken place; (3) what were the characteristics of the lime used? Was it high magnesium or high calcium? Was it lump, pebble or pulverized lime?

Lime putty in closed receptacles is likely to hold its heat for a great length of time and it does not necessarily follow that this heat is caused by the progress of hydration.

I would say that the ordinary 1:2:4 mix would take about one cubic foot of putty per cubic yard of concrete. This means that the cubic foot of putty is about 1/22 of the solids and about 1/27 of the total mass. If the putty were introduced into the concrete at 180° F. and the temperature of the other ingredients was 60° F. it is doubtful whether the total mass would be increased in temperature by more than 5 to 10° F.

If lime putty, even though it be well seasoned, is of stiff consistency when added to the other ingredients, there is very great likelihood of so-called "butterflies" in the final concrete mass. We have had the experience years ago using lime putty which had to be thinned down to a consistency of a very rich cream to prevent butterflies. Recently we were designing a system for the introduction of lime putty into concrete at our Philadelphia years. The design called for a tank where the putty was agitated and by means of the addition of water kept at the consistency of very rich cream. The water content per unit volume could be readily obtained by weighing known quantities of the putty and the knowledge of specific gravity and the unit weight of the lime hydroxite solids. From this tank the putty would be dripped into a measuring tank. The exit from this measuring tank would be connected with the pipe carrying the water supply to the mixer. The measuring of the lime putty and the regulations of the quantity of water would be done by the concrete inspector at the plant.

If the milk of lime is passed through a fine screen and the consistency used is kept constant so that the water content is known and the putty is introduced into the stream of water going into the mixer, I believe the ready mixed concrete manufacturer need have no fear for his concrete.—ALEXANDER FOSTER, JR.*

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Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

The Sieveringer bridge

F. BARAVALLE, *Beton und Eisen*, Vol. 38, No. 6, p. 93 (Mar. 20, 1939). Reviewed by A. U. THEUER

A description of a reinforced concrete grade separation structure in Vienna is here presented with considerable detail.

Road bridges in Great Britain

Concrete and Constructional Engineering, Vol. xxxiv, No. 1-4, Jan.-Apr., 1939, pp. 99-130, 159-190.
Reviewed by GLENN MURPHY

Descriptions, working drawings, and photographs of recent reinforced concrete bridges in Great Britain.

A reinforced concrete bridge over the Rio Tiete River in Brazil

HERMANN SCHROEDER, *Beton und Eisen*, Vol. 38, No. 6, p. 100 (Mar. 20, 1939).
Reviewed by A. U. THEUER

The author describes the design and construction of a 440 ft. span reinforced concrete arch bridge with suspended span. The principal interest here lies in methods of construction made necessary by the isolated location of the building site.

Ferric oxide in cement

A. J. JOHNSON and J. T. LYNN, *Rock Products*, Vol. 42, No. 5, May 1939, pp. 36 and 37 (Concluded in June 1939 number). Reviewed by ROY N. YOUNG

Attention is called to the possible errors in the use of diphenylamine as an indicator in the dichromate titration of ferric oxide as now prescribed in certain standard procedures, and to the advantages of diphenylamine sulfonic acid as an indicator. A recommended method for the use of the latter indicator is given.

Application of pre-stressing in reinforced concrete construction

Bautechnische Mitteilungen des Deutschen Beton-Vereins E. V., No. 3, pp. 1-5, Oct.-Dec. 1938.
Reviewed by F. L. EHASZ

Several structures built by Finsterwalder and Dischinger technique are cited as illustrations of the application of pre-stressing in reinforced concrete construction. Bolder constructions and considerable economy in steel are said to be attributable to these new methods.

Reinforced concrete roofs

E. SUENSON, *Beton und Eisen*, Vol. 38, No. 4, p. 65 (Feb. 20, 1939) Reviewed by A. U. THEUER

This is a summary containing a miscellany of details connected with reinforced concrete flat slab roofs. Types of roofs are classified and details regarding proper drainage, materials for topping, heat insulation, resistance to abrasion, proper provisions for edge treatment, gutters, parapet walls, etc., and for protective measures around roof openings are discussed at length.

Convention of German Portland Cement Association in Berlin

Hauptversammlung des Vereins Deutscher Portland-Cement-Fabrikanten E. V., Berlin, pp. 1-218, Apr., 1938 Reviewed by F. L. EHASZ

Papers presented at the principal convention of the Association of German Portland Cement Manufacturers dealt with the chemical reactions in the hardening of hydraulic cement, the action of fluoride in clinker burning, and observations on a trip to Canada and the United States.

Protective curbs for highway curves

SVEND RASMUSSEN, *Beton und Eisen*, Vol. 38, No. 4, p. 63 (Feb. 20, 1939).
Reviewed by A. U. THEUER

The author describes a precast reinforced concrete curb designed to minimize hazards around dangerous highway curves in Denmark. The curb is so designed that a low protecting edge will contact the tires of vehicles thrown against it and act both as a guide as well as a frictional brake. Observations of the effect of this curb in tests and in actual accidents are described very favorably.

Stress distribution around a tunnel

RAYMOND D. MINDLIN, *Proc. Am. Soc. C. E.*, Vol. 65, No. 4 (Apr. 1939), p. 619-642.
Reviewed by H. J. GILKEY

By considering the tunnel as a horizontal cylindrical hole situated in a semi-infinite elastic solid under the action of gravity and applying the methods of elasticity, the author derives tables and curves which show that the stresses are important when the hole is either very close to the upper surface of the solid or is very far from the surface.

Concrete girder carries 800-seat balcony

W. T. WRIGHT, *Engineering News-Record*, Vol. 122, No. 3, Jan. 19, 1939, pp. 66-67.
Reviewed by R. W. BEAL

Details are given for the construction of the 9 ft. deep, 85 ft. span solid concrete girder used as the main support of a balcony in a California school building. The tensile reinforcing bars were plain, 2-in. square and continuous, no splicing being used. The girder sections varied from rectangular at the ends to T-shaped at the center. The girder was supported on timber piling during construction.

A new application for asbestos-cement

PATRICIO PALOMAR, *Tonindustrie Zeitung*, Vol. 63, No. 25, p. 296 (Mar. 27, 1939).
Reviewed by A. U. THEUER

Author comments on the increasing use of asbestos-zement (Eternit, Uralit, etc.) in industrial applications. He then describes its use for rollers under an aqueduct crossing a large stream in Spain. The method of manufacture (Mazza-Spanish Patent) as well as the physical properties of this material are briefly described in a very general manner. Compressive strengths up to 18,000 p.s.i. and tensile strengths up to 2500 p.s.i. are claimed for the material.

Graphical arch analysis applicable to arch dams.

CARL H. HEILBRON, JR. and WILLIAM H. SARLOR, *Proc. Am. Soc. C. E.*, Vol. 65, No. 1 (Jan. 1939), p. 3-27. Reviewed by H. J. GILKEY

The detail procedure with an illustrative example is given for analyzing unsymmetrical fixed-end arches of variable thickness and radius under loads of any type. After writing the general arch analysis equations, approximate deflections and stresses in the arch may be obtained. The equations are then modified to include the concept of "trial crown thrust" and by the graphical technique shown, the stresses and deflections may be refined to the degree desired.

Thin concrete shell roof for ice skating arena

ANTON TEDESKO, *Engineering News-Record*, Vol. 122, No. 7, Feb. 16, 1939, p. 54. Reviewed by R. W. BEAL

The thin-shell arch roof of an Ardmore, Pennsylvania ice skating arena covers a plan area of 105 x 235 ft. The structure consists of a number of monolithic arch and shell units; each unit is comprised of two arches on 29 ft. centers, with the shell connecting the two arches and cantilevering 9 ft. beyond each arch. The spaces between the cantilever ends of adjacent units allow for corrugated wire glass skylights. The cost of the structure was $3.00 per sq. ft. of plan area or 10 cents per cu. ft. of building.

Concrete superstructure on steel bridge

Engineering News-Record, Vol. 122, No. 13, Mar. 30, 1939, pp. 49-50. Reviewed by R. W. BEAL

On the Pius IX Boulevard Bridge at Montreal two hinged steel arch ribs were used to support a concrete superstructure. Reinforcing for the concrete superstructure was of welded construction and was not only self-supporting but carried the construction loads. The hollow steel ribs of 103-ft. unsymmetrical end span were filled with concrete for balance and added thrust strength. The steel ribs were painted gray, and, while the paint was still wet, were sprayed with a fine white sand to give an appearance very similar to the concrete of the deck system.

Concrete forms and the four year plan

C. KUPFER, *Beton und Eisen*, Vol. 38, No. 5, p. 77 (Mar. 5, 1939). Reviewed by A. U. TREUER

The author discusses the factors involved in detailing concrete forms to provide for maximum economy and efficiency in the use of materials and labor. His principal recommendation is for the use of a standard (width and thickness) plank throughout all work. In this he favors the use of a plank 10.5 by 2.5 cm. (4.15 by .095 in.) which is now in common use in North Germany. This plank is said to provide a very satisfactory cross section for the various purposes, facing, bracing, etc., required in a complete formwork for concrete building structures.

Design of circular concrete tanks

GEO. S. SALTER, *Proc. Am. Soc. C. E.*, Vol. 65, No. 3 (Mar. 1939), p. 419-428. Reviewed by H. J. GILKEY

By a rigorous mathematical analysis based on the assumption of fixity between wall and base and dividing the internal liquid pressure between the circular rings and the vertical cantilevers, the magnitude of the maximum ring tension in a circular tank may be reduced to 25 or 30 per cent of that assumed in a tank not considered fixed at the base. For tanks of the common proportions of those used for settling purposes, the amount of wall reinforcement required may be reduced as much as 50 per cent. Diagrams and an illustrative problem are given to facilitate the application of the method.

Making and testing of 4 by 4 by 16 cm. standard test prisms

Tonindustrie Zeitung, Vol. 63, No. 26, p. 312 (Mar. 30, 1939). Reviewed by A. U. THEUER

Recent changes in the German standard specifications for highway cements include the following. Tensile strength tests are replaced by a flexural strength test made on 4 by 4 by 16 cm. bars. An artificially graded sand is substituted for a uniform test sand and a shrinkage-drying test is henceforth to be included.

The author gives the details of the manufacture of specimens and methods of making the flexural strength tests together with a number of comments. The procedure to be followed in making the shrinkage test is also included.

Bus and coach garage in central London

Concrete and Constructional Engineering, Vol. xxxiv, No. 3, Mar. 1939, pp. 149-158.
Reviewed by GLENN MURPHY

Reinforced concrete construction was used in a new garage built by the London Passenger Transport Board. The structure is to accommodate 75 motor coaches in the basement, 100 double-deck busses on the ground floor, and to have offices on the first floor. Slab-beam-girder construction was used, the girders being 44 in. deep with spans of about 48 ft. Two intermediate 26 in. beams are used in each bay to support the 6 in. slab. A working stress of 1100 p.s.i. was used for the concrete, and Isteg bars served for the reinforcement. The design includes a ramp for moving busses from one level to the other.

Convention of German Portland Cement Association in Vienna

Wanderversammlung des Vereins Deutscher Portland-Cement-Fabrikanten E. V., Wein, pp. 1-220, Sept. 1938. Reviewed by F. L. EHASZ

Reports given at the Vienna convention of the Association of German Portland Cement Manufacturers included the following topics: mineral wealth and construction economy of Ostmark, new methods of reinforced concrete construction, reinforced concrete structures with a minimum amount of steel and the highest compatible carrying capacity, Rontgen investigations of cement, determination of setting heat of cement, activity of the research committee, and shrinkage and flow of portland cement.

Determination of the drying shrinkage of cement mortars—a small-scale method

R. W. NURSE, *Journal of Chemical Industry*, Jan. 1939, Vol. LVIII, pp. 37 and 38.
Reviewed by R. W. CARLSON

A method is described for determining the volume change of small specimens of cement mortar by weighing them when submerged in mercury. Results of tests by this method on 1-inch cubes are compared with results of tests by the usual method on larger prisms. It is found that the ratio of linear shrinkage of 2 by 2 by 12-in. prisms to volume shrinkage of 1-in. cubes is practically constant for various mixtures and equal to 0.15. Thus the "mercury-immersion" method employing small specimens appears to be useful where only small quantities of material are available for test. It may also provide the basis for an accelerated test.

Estimation of phase composition of cement clinker

L. A. DAHL, *Rock Products*, Vol. 42, No. 4, Apr. 1939, pp. 50-52. Reviewed by ROY N. YOUNG

This is the seventh and concluding article of a series under the above title that was begun in the Sept. 1938 number of *Rock Products*. In the several articles the author explains the procedures used in his mathematical treatment of phase equili-

bria particularly as applied to the system $3CaO.SiO_2$—$2CaO.SiO_2$—$3CaO.Al_2O_3$—$4CaO.Al_2O_3.Fe_2O_3$ at clinkering temperatures. A new classification of composition types is used and diagrams are shown for determining the composition of liquids for the respective types at different temperatures. Equations are given for a simplified method of calculating (1) phase compositions at points in the process of fusion, (2) products of complete independent crystallization, and (3) phase composition of clinker with an estimated glass content after normal or partial independent crystallization.

An investigation of rigid frame bridges

Part I. "Tests of Reinforced Concrete Knee Frames and Bakelite Models," by F. E. Richart, T. J. Dolan and T. A. Olson. Bull. 307, Eng. Expt. Sta., Univ. of Ill.

Part II. "Laboratory Tests of Reinforced Concrete Rigid Frame Bridges," by W. M. Wilson, R. W. Kluge and J. V. Coombe. Bull. 308, Ibid. Reviewed by F. E. Richart

Part I gives the results of rapid and long-time tests of reinforced concrete knees, representing corner details of a rigid frame bridge. The tests are supplemented by photo-elastic studies of bakelite models.

Part II contains a study of stresses due to load and to temperature change in two rigid frame bridges of 48-ft. span and 16-ft. height. The results of tests of these bridges to failure are also included.

Portions of both bulletins have previously been presented to the Institute as papers, see A.C.I. Journal (I) March-April, 1937, pp. 459-479 and (II) May-June, 1938, pp. 625-648.

Proceedings Highway Research Board

The proceedings of the 18th Annual meeting, Highway Research Board are announced in several parts—separate prints of Reports of Roadside Development Committee and Discussion on Rural Traffic Problems and Vol. 18 in two parts. Part I includes papers on Finance, Economics, Design, Materials and Construction, Maintenance, Traffic and Safety, Soils—the last named item including only digests of papers covered fully in Part II which includes papers on Soils Mechanics, Fill Compaction and Stabilization with a compendium on Soil testing. A few of the papers in Part I, more especially in the A. C. I. field are: "Strain in Bond Pullout Specimens," H. J. Gilkey, S. J. Chamberlain, R. W. Beal; "Impact on Reinforced Beams," T. D. Mylrea; "Dowels in Transverse Pavement Joints," Bengt Friberg; "Map-cracking in Sand-Gravel Concrete Pavements," W. E. Gibson; "Deleterious Materials in Concrete Aggregates", F. C. Lang; "Maintenance Costs," H. K. Bishop; "Concrete Pavement Warping," Harold Allen.

Tunnel paved with concrete blocks

Engineering News-Record, Vol. 122, No. 15, Apr. 13, 1939, pp. 62-64. Reviewed by R. W. Beal

Precast concrete blocks are being used to pave the 178th St. underground vehicle approach to the George Washington bridge in New York, Some form of removable block pavement is considered necessary because of the possibility of excess wear due to the close tracking of heavy traffic in tunnels. The cushion for the blocks is a 1⅛ in. layer of hot-mixed mastic made from 93 to 96 per cent dry sand and 4 to 7 per cent cutback asphalt. Each block is 4 in. thick and 11¾ in. square, weighing about 50 lb. The blocks are staggered in transverse courses with ⅛ in. to ¼ in. joints all around, asphalt being used as the joint filler. One inch boards layed longitudinally on the newly placed blocks provide a runaway for a 5-ton tandem roller, which embeds the blocks into the mastic about ⅛ in. The 28 day compressive strength of the blocks is in excess of 6000 p.s.i., and the paving is costing about $3.09 per sq. yd., exclusive of overhead.

New devices for curb construction

ROBT. M. ANDREW, *Engineering News-Record*, Vol. 122, No. 15, Apr. 13, 1939, pp. 44-46. Reviewed by S. J. CHAMBERLIN

A new dual pavement, consisting of two 22-ft. roadways with 2-ft. lip gutters and a 4-ft. raised center strip, tops and widens an old 30-ft. concrete road. Instead of using a face form for the top of the gutter a strikeoff 4-ft. long equipped with a vibrator was used. The straight edging was guided by templets made of ½ in. tees. The dividing strip consists of two 12-in. curbs with 2 ft. of black concrete between them. The center curbs consist of a colored 1-in. top on a plain concrete base. A wheeled strikeoff pulled by the mixer shaped both base courses at once. The topping was struck and finished with a special hand tool. The yellow top was finished without difficulty excepting unusual care had to be exercised to secure good line with a trowel finish. The white topping with local aggregate gave the best trowel finish. Curing paper protected the pavement while the work on the curb progressed and was used in curing the finished curb as well as for protecting the curb while the black center strip was placed.

Grouted ballast slab for track support

Engineering News-Record, Vol. 122, No. 15, Apr. 13, 1939, pp. 56-57. Reviewed by S. J. CHAMBERLIN

The Pennsylvania R. R. has constructed two test sections of sub-ballast, grouted slab to support track where ground conditions were bad. At the first section after the track and ballast were removed the clay was dug out to a depth of 2 ft. below the bottom of ties, side forms were erected and regular crushed stone ballast filled in and tamped. Grout from a hopper tank on the adjacent mixer train was fed by gravity through 50 ft. of 2½ in. fire hose to the stone. Excess grout was brushed ahead. The grouted stone was again tamped and left with an unfinished surface to help retain the ballast. Ties were placed on the slab within 3 days, the rail replaced, the track jacked for a 12-in. raise and ballast added. At the second section the track was not taken out of service, sections of two rail lengths being rebuilt in 7 hours. It proved advantageous to increase the consistency of the grout to produce a flow of 23 to 24 secs. After 10 to 12 ft. of the stone had been grouted the ballast was dropped on it, spread, and the track replaced. Grouting into stone filled with water to the top of forms at some places showed no visible ill effects.

Investigation of the compressive and flexural strengths and permeability of lime mortars.

OTTO GRAF, *Tonindustrie Zeitung*, Vol. 63, No. 27, p. 320 (Apr. 3, 1939). Reviewed by A. U. THEUER

This article describes, with the aid of tables, an extensive series of tests of different brands of lime combined with different types of sand and different sand gradings. Test specimens used were 4 by 4 by 16 cm. prisms. The location and number of cracks, density, flexural, and compressive strengths, height of rise of capillary water at 10, 30, 60, 120 and 180 minute intervals, and total quantities of water absorbed in 5½, 24 and 48 hours were measured. Special observations were made in the case of a limited number of specimens.

In practice, mortars are subdivided into three classes: (a) outside finishes, (b) inside finishes and (c) masonry joint mortars. For (a) the author recommends the use of coarse sands (maximum size 7 mm.), and for (b) and (c) fine sands (maximum size 3 mm.). The gradings of sand for (a) and (c) must be such as to result in an impermeable mortar having a satisfactory strength. Both workability and thickness of finish coatings must be carefully watched in practice.

Measuring dam behavior

DOUGLAS MCHENRY and ROY W. CARLSON, *Engineering News-Record*, Vol. 122, No. 13, Mar. 30, 1939. pp. 58-60. Reviewed by S. J. CHAMBERLIN

Temperature, strains, stresses, joint openings, uplift pressures, foundation deformation and horizontal deflections are being measured on dams to check on design assumptions. Measurements of uplift at Norris Dam with pressure cells have shown that the assumption of linear variation from full head at heel to zero at toe is sufficiently conservative and have shown the effectiveness of relief holes. Deflection measurements made by means of a stainless steel plumb line and measuring microscopes, indicate that the deflection due to the annual cycle of temperature is roughly equal in magnitude to the load deflection for full reservoir height. Interior strains were measured by elastic wire strain meters embedded in the dam. Stress-free specimens for control showed greater length changes than would be computed by using the coefficient of thermal expansion, indicating a growth of the concrete under moist conditions. Compressive stresses expected because of the initial temperature rise in the concrete evidently did not develop because of the plasticity of the green concrete.

Determination of specific surface of powders, particularly of portland cement

HANS-JOACHIM HARKORT, *Forschugsarbeiten aus dem Strassenwesen*, V. 15, pp. 1-61, 1939. Reviewed by F. L. EHASZ

This paper offers a critical survey of various methods employed in ascertaining the specific surface of portland cement. In the introduction it is stated that the common measure of fineness in terms of sieve numbers has been superseded in modern research by more accurate means. It is felt that a true picture of the reactivity of a cement is to be obtained only through a determination of the effective surface. The task of finding the most expedient method of those recommended is not an easy one. Indirect methods of "absolute" surface determination on the basis of calculation from the particle size distribution make use of the sedimentation balance of S. Oden the pipette analysis, and the hydrometer; direct methods, on the other hand, employ air separators, the sedimentation analysis of Atterberg, and the Kuhl-Czernin sedimentation apparatus. Klein's suspension turbidimeter, and Wagner's turbidimeter are mentioned as being instrumental in the "relative" surface determination by optical means. Many references on these methods are available in the extensive bibliography.

Studies in porous materials with special reference to building materials. I. The crystallization of salts in porous materials

D. G. R. BONNELL and M. E. NOTTAGE, *Journal of the Society of Chemical Industry*, Jan. 1939, Vol. LVIII, pp. 16-21. Reviewed by L. R. BORBRICH

It has long been known that certain building materials may be disintegrated by the crystallization of salt hydrates within their pores. Although it has been assumed that this disintegration is associated with the volume change accompanying hydration, no experimental evidence has been available in support of this contention. This report describes some experiments made with this purpose in view. The general procedure consisted in measuring the changes in volume which occurred as a result of the hydration of salts in a porous body of little or no tensile strength, under known applied loads. From the results on measurements using hydrates of sodium and magnesium sulfate the authors concluded that anhydrous salts (or the lower hydrate) may be hydrated (or further hydrated) even against moderately high stresses, and

since the stresses exerted by the salt hydrates are well above the tensile strength of normal building material, it is possible that during its hydration a salt may exert sufficient force to disintegrate the material. The bearing of this behavior on the differentiation of building materials by the crystallization test will be discussed in a paper which will be published shortly.

The Farjsund bridge, Finland

Concrete and Constructional Engineering, Vol. xxxiv, No. 2, Feb. 1939, pp. 95-97.
Reviewed by GLENN MURPHY

Design and construction features of a reinforced concrete arch bridge having a span of 426 ft., a rise of 58 ft. 6 in. and two reinforced concrete girder approaches, each consisting of two spans of 49 ft. and 44 ft. The bridge carries a 16 ft. 6 in. roadway and two foot paths. The arch rib was designed as a twin-cell box section, consisting of an upper and lower slab joined by three vertical longitudinal diaphragms. The section is 3 ft. deep and 15 ft. wide at the crown, and is 8 ft. 6 in. deep and 19 ft. wide at the springing. The top and bottom slabs vary from 10½ in. to 13 in. in thickness, the top slab forming the roadway surface in the middle quarter of the span. The arch was designed to carry a 6-ton truck and a uniform load of 70 lb. per sq. ft., or a 10-ton truck alone. Finnish regulations require consideration of a temperature range from 27 °F. to -54 °F. The design stress was 1020 p.s.i. for the concrete in the ribs, 820 p.s.i. for the concrete in the remainder of the structure and 1700 p.s.i. in steel.

During construction a gap of 24 in. was left open at the crown for two months after the rest of the rib had been cast. Jacks were then inserted in the gap and pressure applied to compensate for the rib shortening due to shrinkage and compression. When the arch had been adjusted to the proper position, the gap was filled with concrete, and the jacks subsequently removed in the usual manner.

Investigations of concrete and mortar strengths

F. KEIL and F. GILLE, *Tonindustrie Zeitung*, Vol. 63, Nos. 18, 19, 20, p. 198, Mar. 1939.
Reviewed by A. U. THEUER

An investigation into the relation between laboratory mortar and field concrete strengths is here described. The series of tests carried out as part of a larger investigation undertaken by different laboratories was initiated as an answer to the following specific questions: (1) What is the influence of compaction on the strength of mortars containing different water-cement ratios; (2) What strength relation exists between concretes and mortars of equal wetness over a large range of wetnesses; (3) What effect does different curing treatments have on mortars of equal strength and wetness. The conclusions arrived at after a detailed description of the methods used and an analysis of the data are as follows:

(a) Vibration serves as an aid in compacting laboratory test specimens of low wetness, resulting in greater uniformity of test results. Author recommends thorough rodding followed by vibration with superimposed loads. In the case of wet mixes, no advantages were found with vibration.

(b) With water-immersed or damp-cured specimens a well-defined relation between the compressive strengths of mortar and concrete specimens of equal wetness was found. (Water-cement ratio 1:6.2). To a somewhat lesser degree a similar relation was found for flexural strengths.

(c) With mixed curing well defined changes were charted for the flexural strength of concretes, but not for the mortar specimens.

(d) For the cements used in this investigation, only the results of strength tests made on the damp-cured mortar specimens were indicative of their effects on concrete strengths.

Czech Institute of Research

Vyzkumny a zkusebni ustav hmot a konstrukci stavebnich pri Ceskem vysokem uceni technickem, pp. 1-55, 1938. Reviewed by F. L. Ehasz

The Institute for Research and Testing of Materials and Structures, which is an important part of the Czech Technical University governed by the Ministry of Education, was founded in 1921 with a view to fostering scientific and educational endeavor in the field of applied mechanics, to giving lectures and courses, and to serving the public authorities, private enterprises, manufacturers and consumers.

Listed within the scope of its program are the following, taken verbatim from the article: (1) building stones, (also sand and gravel) natural and artificial; (2) cements, limes and plasters, concrete and other mortars to be used in building, water power, highway, bridge and similar structures; (3) asphalts, tars and the like; (4) timber and all kinds of wood for special purposes; (5) steel for buildings and mechanical and other metals; (6) all structures and products made of the previously mentioned materials, chiefly in actual size and shape, i.e., slabs, beams, walls, columns, floor and ceiling structures, rigid and other frame structures, etc.; (7) investigation of foundation soils in the laboratory and on building sites; (8) investigation of completed structures in the field regarding their behavior when loaded or heated, their noise transmission, etc.; (9) investigation of structures represented by models; (10) re-testing and certifying of measuring and testing devices; (11) collaboration with the authorities and properly authorized bodies when standards and other important rulings are made which may affect the use or control of building materials of every description; (12) consulting and professional services in court.

Plans of the three-story laboratory layout and some of the machinery and equipment used in current investigations are included; among those utilized for structural problems are testing machines for maximum loads of 500 and 1000 tons, others for maximum compressive or tensile load of 250 tons with a pulsator for maximum load of 100 tons and 500 reversals per minute, bending test machines for the testing of concrete pipe and of floor slabs with heating pipes "Crittal" embedded in place of regular reinforcement, and equipment for the testing of a model vaulted shell of concrete and glass. From this article it would seem that the Institute is well equipped for testing properties of materials and investigating structural behavior.

Variation in strength of concrete test cubes

Nils Tengvik, *Cement och Betong*, Feb., 1939, p. 15. Reviewed by Bengt Friberg

The paper presents a statistical survey of 2800 series of concrete test cubes, tested at official Swedish testing laboratories during the period of Spring, 1935, to Spring, 1937; 700 tested at 7-day age, 2100 tested at 28-day age, each series including 3 cubes with 20 cm. side. Bridge building concretes included those with 300 and 350 kg. standard cement per m^3.

Ninety per cent of the bridge concrete series tested above the required strength of 130 and 165 kg./$cm.^2$ for 7-day tests, and 220 and 275 kg./$cm.^2$ for 28-day tests. Excluding 10 per cent of the series with abnormally high strengths, the 7-day tests go up to 26 per cent and 233 per cent; the 28-day tests up to 219 per cent and 190 per cent of the required strength for concrete with 300 kg. and 350 kg. cement per m^3,

respectively, an increasing uniformity in strength with increasing strength. The concrete containing 350 kg. cement per m³ was uniformly about 50 kg. per cm.² stronger than the concrete with 300 kg. cement per m³ both at 7-day and 28-day age for any percentage of all series.

The 7-day strength is commonly rated as 60 per cent of 28-day strength. The ratio between 7-day and 28-day strengths varies for different cements from .50 up to .70 or .80. The ratio increases with increasing 7-day strength. In this survey 198 double series of 6 cubes, 3 tested at 7 days, 3 at 28 days, show strength relations according to formula: $f_7 = \left(40 + .1 f_7\right)\%$ of f_{28}; (Strengths in kg./cm.²). The 7-day tests give sufficiently close indication of strength to justify curtailment of 28-day tests in favor of 7-day tests.

Maximum per cent deviation between individual test and average strength of each series of three is for 50 per cent of the series less than 4 per cent; deviations greater than 10 per cent occur in 10 per cent of the series. The curve for maximum deviation in each series plotted against percentage of all series is the same for both 7 and 28-day tests. The deviation decreases with increasing strengths for both 7-day and 28-day tests.

The observations indicate that from the point of view of strength values and uniformity, an increased factor of safety should not be required with increased design strength of concrete.

Collapse of a reinforced concrete arch structure

F. V. EMPERGER, *Beton und Eisen*, Vol. 38, No. 6, p. 112 (Mar. 20, 1939). Reviewed by A. U. THEUER

The author describes the structural design, methods of construction and pertinent facts regarding the partial collapse of a reinforced concrete arch-knee framed structure housing a long tunnel oven. As originally designed 34 3-hinged arch-knee frames were to support the roof of the structure. Mesnager type hinges were used, one horizontally at the crown of the arch and the other two vertical, joining the vertical leg of the frames to the footings on either side.

The job was carried out in two halves. The first half consisting of 17 bents was satisfactorily completed. Hair cracks along the arches appeared soon after stripping the forms, but little or no study was given to them. The second 17 bents collapsed immediately after the forms were stripped. In the failure the legs on opposite sides of each bent were sheared off and pushed outwards near the footings.

A careful check-up of the first half of the structure was not made until after failure. Measurements of the center deflection of these arches after the failure revealed a 4 to 6 in. drop. A somewhat general discussion regarding the significance of the maximum deflection in structural work follows. Little attention other than the making and careful recording of deflection measurements has followed in practice. Serious attempts at interpretation have never been attempted. With the adoption of high strength steels further significance is given to these measurements. After years of successful practice in steel construction, the author comments on the increasing of the value of the maximum deflection of 1/1000 of the span up to 1/600. In order to further facilitate the use of high strength steels proposals are now being made without any technical backing to further raise this value to 1/300 of the span.

In the case of this particular study using conventional design formulas, a center deflection of 5.3 in. or 1/190 of the span is indicated. This figure is stressed not only to show how much larger than usual in practice is the deflection here involved, but also to indicate how much allowance must be made in construction for proper centering. In the case of this structure, no provision whatever was made for this factor.

The following factor as an element to explain in part the collapse is then developed. A 2-in. clearance for the hinges was called for in the design. To obtain this clearance the contractor used "Heraklith-plates" to this total thickness stacked up on the footings at the proper point. The forms were then built and concrete poured in a continuous schedule until completion of the bent. From appearances it is judged that this material was not sufficiently water-resistant, became water-logged and soft and under the pressure of the fresh concrete was compressed to a final thickness of from 0.4 to 0.8 in. That this factor alone caused failure cannot be maintained, however, since it holds for both the part of the structure still standing as well as for the part that has collapsed. The concrete covering the hinged joints and the joints themselves were too completely destroyed to indicate just what forces were active at failure. The fact that the vertical hinges have locked in that part of the structure still standing might be taken to prove that displacements of this magnitude can be equalized within the structure itself.

Since provisions for wind loads were neglected, the horizontal thrust developed its maximum force at the hinged connections following the removal of forms. That this possibility was unforeseen by the designer is indicated by insufficient spiral reinforcement in the footings surrounding the hinges, even though the hinge frames themselves were adequately tied together.

The authors moral here is that designers should more carefully consider just what the effect on their designs will be if errors possible in construction should occur.

A contribution to the question of bond stresses in reinforced concrete

E. Hoyer and E. Friedrich, *Beton und Eisen*, Vol. 38, No. 6, p. 107 (Mar. 20, 1939).
Reviewed by A. U. Theuer

The observation in practice of the lack of any tendency for small steel wires highly prestressed and used as reinforcement to slip or be drawn into incasing concrete following the release of prestress and in the absence of end anchorage is made the occasion of a general discussion of bond. With the successful application in practice of "steel-wire-concrete" (high strength concrete reinforced with steel wires .08 to .16 in. in diameter and prestressed up to as high as 175,000 p.s.i.), a reconsideration of the problem of bond is considered necessary.

In a lengthy introduction, current design specifications bearing upon bond are characterized as nothing more than a general and classified repository for the experience of 50 years of reinforced concrete practice. An adequate solution of the complex problems centering on bond (including cracking) is considered to be still forthcoming. With the introduction of high strength reinforcing steels and new materials (14,000 p.s.i. concretes), new problems are posed that cannot be covered by past practice. The first result of the introduction of high-strength steels has been a variety of new barcross-sections (Isteg, Drillwuhl, etc.). With the additional observation following from the practice of steel-wire-concrete, the inability of current specifications is further evidenced.

Taking up the subject of bond the authors then present an analysis to indicate the nature of stress transfer from steels to concrete and just what the nature of the resulting distribution of compression in the surrounding concrete envelope will be. This relation is found to follow a hyperbolic law, considering a single bar only. With the substitution of many bars for a single one, greater uniformity of stresses over a concrete section follows. Actual quantitative evaluations are not attempted.

In a second part of the analysis the authors take up the subject of end anchorage. The opinion often met in practice that increasing length of imbedment of rods will

increase total bond strength and eliminate end anchorages is first disposed of. The authors then emphasize an important distinction arising from the diameter of bars. Below a certain critical diameter the tensile strength of a bar will always be less than its bond strength. Above this critical diameter the bond strength is lower than the tensile strength of the rod and at any point where this condition holds bond will be destroyed. Increasing the length of bar changes nothing. Bond will be progressively destroyed along the full length and end anchorages become necessary. For the steel wire under discussion this critical diameter has been roughly set at 0.16 in. A theoretical analysis of this phenomenon may be briefly summarized as follows:

With an original diameter of *Do* the wire will, on release of prestress, have a smaller diameter *Dx* at some section removed a distance x from the end. The change in diameter at section *Dx* is assumed to be proportional to the frictional force and the differential equation of the shape of the wire will then be

$$\frac{dDx}{dx} = A\ (Do - Dx)$$

A being a constant of proportionality and the type of equation indicating an e-potential relation. Diagrams illustrate the nature of the variation of the compressive stress in the concrete eveloping the bar, the bond stress along the bar, and the tension in the bar. Bond accordingly is made a function of the diameter of the bar only just as long as the compressive strength of the concrete is not exceeded.

Without prestressing and in the case of a large-sized bar a different phenomena follows. At the points of stress transfer from steel to concrete large bond stresses occur which are proportional to changes in the tensile stress of the bar. (A theoretical expression is here given.) As soon as the bond stress becomes greater than the bond strength "at any point" slip will there occur. The tension over the free length of bar remains constant. At the point where bond is still effective a sharp decrease in tension occurs. Since the bond stress is proportional to change in tension, a large bond stress will occur at this hump. Since the maximum bond force can be less than the maximum tensile force, slippage will progress along the full length of the bar. By means of diagrams, the stress distribution over the cross section of the concrete envelope is shown to be far less uniform than is the case with a bar of small diameter.

The subject of temperature is touched on briefly because of the recent applications of steam curing in practice. This practice is considered as inadmissible in reinforced concrete work because of the expansion of the steel when subjected to heat with consequent shrinkage resulting in a loss of grip.

(In the current issue of *Bautechnik*, Vol. 17, No. 10, p. 128, an article describing several recent steel-wire-concrete girders is to be found).

The author calls for a more complete study of creep and shrinkage effects resulting from prestressing.

Structural bending moments simplified

ULRIK T. BERG, Polygraphic Co. of Am., 186 pp., obtainable from the author, Brooklyn, N. Y. ($4.50)
Reviewed by C. W. Ross

The analytical methods of the book are simplified extensions of the Ritter-Strassner graphical methods. The author first finds the factors which give the relationships between the moments in one member and those induced in adjacent members. Then, by the aid of these factors, he distributes throughout the structure the moments produced by the loadings.

A moment in a continuous structure will produce rotation and bending in the neighboring joints and members. These are proportional to the moment and, if the

factors of proportionality are found, the rotation and bending due to moments of another system of loading can be found easily.

The author's first step is to find the stiffness of the individual members. For members of uniform section and elasticity, this is done in the usual way. In the case, so frequently met with in reinforced concrete structures, in which the ends of the members are not of the same size (haunched members), their stiffness can be expressed as the stiffness of a similar member of uniform section multiplied by a constant. The three basic constants are S_A, S_B, and t equal to

$$\frac{1}{\frac{2}{L^2}\int_o^L \frac{L-X}{I_x/I}\,dx}, \quad \frac{1}{\frac{2}{L^2}\int_o^L \frac{x\,dx}{I_x/I}}, \text{ and } \frac{6}{L^3}\int_o^L \frac{(L-X)x\,dx}{I_x/I}$$

respectively, in which I_x is the moment of inertia at point x; I is the moment of inertia of the minimum section of the member; and L is the length of the member. If there is a uniform moment throughout the length of a member, the rotation of the ends is given by $\phi_A = \frac{M}{2KS_A}$ and $\phi_B = \frac{M}{2KS_B}$

in which $K = \frac{EI}{L}$ being the modulus of elasticity. If there is a moment acting at one end only of an unrestrained member $\phi_A = \frac{Mt}{6K}$, in which ϕ_A is the rotation of the end opposite to that on which M acts.

For convenience in computation either these constants or others derived from them are tabulated for a large variety of shapes, and a method is given for the treatment of members of more complicated shapes by dividing them into parts.

If a couple is applied at one end of a member, there will always be a section at which the bending moment of the member will be zero. This section is the same regardless of the magnitude of the couple applied providing the proportional limit of the material is not exceeded. If there is no restraint at the far end of the member, this section will be at that end. If there is restraint, due to the action of other abutting members, this section will be at a distance from the end. The author uses this distance as a measure of the restraint. The distance of the section from the end opposite to that on which the couple acts is written UL: similarly for the other end of the member there is a section at a distance VL from that end which is a measure of the restraint at that end. The values of U and V for a member can be found from the sum of the stiffnesses of the other members, and very easily by means of a table giving the values of the product UF and VF for a wide range of values of the restraint, where F is a derived constant equal to $\frac{1}{St}$.

A large variety of problems are solved by the aid of these constants and others derived from them. For a few problems other special constants and fixed points are introduced to make the solution more simple.

Framed structures of a large variety are treated, such as multiple-span bents, arches, rectangular frames, and rigid-frame bridges, in both the theoretical part and among the numerous examples.

The methods for considering the effects, of various types of loads, temperature deformations, and movements of the joints or supports are given.

The computations are greatly simplified by the numerous tables and graphs of the relations of the different constants to each other and to the dimensions of the

members of the structure. The author puts the more useful relationships in tables which will be found to be of considerable aid to the eyes.

The author discusses also the behavior of columns and beams at their junctures, sign of the bending moments and rotations, rigidity of foundations, and the relation of his methods to "Building Regulation for Reinforced Concrete," American Concrete Institute, of which he reprints paragraphs 601, 701-708, and 1108.

The data given in the book should be of great assistance to designers of continuous structures, especially when the structures are composed of members which are not of uniform section along their lengths. Probably it would not be of value to those who design such structures at rare intervals as familiarity with the methods and data is required for rapid use.

Highway research affecting aggregates

F. H. JACKSON (U. S. Bur. Public Roads) at Convention National Sand and Gravel Assn., Jan. 1939.
Reviewed by STANTON WALKER

The treatment of the problem of aggregate specifications has followed almost a complete cycle. Several years ago, when the importance of controlling the quality of such naturally occurring materials as sand, gravel, and crushed stone first became manifest, it was common to ignore the wide differences in the quality of the basic raw materials and to write more or less arbitrary requirements based on past experiences. Economic considerations soon forced a change in that practice. It became the custom more and more to build specifications around the kind of material most readily available. The pendulum started swinging in the opposite direction. It has the author feels, swung too far and this may account for the present tendency to return to the practice of specifying what is required without regard to local considerations, thus completing the cycle. There is, however, this difference. Whereas the original practice was largely due to ignorance resulting in many cases in absurd requirements, the present trend, is the result of a well-defined feeling among engineers that more rigid requirements are necessary, even though it may mean the elimination of local materials in certain cases and the more thorough and consequently more expensive processing of materials in others.

The premature failure of many structures built in the days when the slogan was "concrete for permanence" has convinced the engineer that more care is necessary in connection with every phase of the art—from the preparation of specifications for the constituent materials to the building of the structure itself. This is true of concrete as used in all types of highway structures, although recently more publicity has been given to the failure of pavements than to other types.

Control of Gradation—It has been common practice for many years to specify size or gradation requirements for sand and gravel. Due to wide differences in the gradation of material in different deposits, it has been necessary to allow comparatively wide variations in the sizes specified. While these wide ranges may be necessary in a general specification, they are entirely too wide when applied to the product of a single plant. For this reason it is becoming the practice to insert what is known as a uniformity clause in sand specifications. The requirement that sand from a given source shall not vary in fineness modulus more than .020 either way from that of a representative sample is an example. The advantages of such a clause from the standpoint of concrete control are obvious. We all know that day-to-day variations in sand grading during construction affect the amount of water required to maintain a given consistency. More precise control of the grading of coarse aggergate is also on the way. Several States are now requiring that the coarse aggregate be shipped

and stored in two separated sizes. Such a step, the author feels, is a distinct advance on the road to more accurate control of concrete quality.

In the case of sand there is a well defined tendency among engineers to require more material passing a No. 50 sieve than has been customary in the past. Engineers feel that the increased workability insured by the presence of from 10 to 15 per cent of this particular size in the mix will much more than offset the slight loss in strength which may result. This question has become more critical with the introduction of cements having high early strength characteristics with the consequent pressure to reduce the cement content. As the concrete becomes leaner it also becomes harsher and troubles from segregation, bleeding, etc., multiply. Fines in the sand go a long way toward counteracting these tendencies.

Experience in some of the Southeastern States, indicate that exceptionally good results may be secured with sands which are so fine that they would not meet even the liberal requirements of the typical general specification. Reference is to sands with a fineness modulus of about 2.0 instead of the more usual 3.0. In the design of concrete mixtures involving such sands the proportion of sand to total aggregate is greatly reduced, which compensates somewhat for the high water demand of the finer sand. The result is a reasonably rich mortar in an amount only slightly in excess of that required to fill the voids in the coarse aggregate.

Control of Quality—In recent developments in specifications from the standpoint of more closely controlling the quality of sand and gravel, efforts are ordinarily centered on the question of insuring durability. In spite of its imperfections, many of the States have adopted some form of accelerated soundness test, either sodium or magnesium sulphate. In many instances these requirements are supplemented by freezing and thawing tests, while in some jurisdictions the results of soundness are waived in cases where the aggregate can show a satisfactory service record. In spite of the work which has been done on this problem, engineers are still quite uncertain as to the real significance of soundness test results. Attempts to correlate the results of tests with service behavior have not proved successful, due not only to inadequacies in the test procedure itself, but also to the influence of many other factors in addition to the aggregate on the durability of concrete. The durability of an aggregate in service depends as much on the kind of concrete in which it is used as on the material itself.

The abrasive resistance of gravel for use in concrete aggregate is of less significance than its durability or soundness. Except in extreme cases, a coarse aggregate which will be satisfactory from the standpoint of soundness will be sufficiently resistant to wear for concrete pavement construction.

Most gravel deposits contain varying amounts of distinctly inferior or deleterious materials. These must be eliminated and the industry should be congratulated on the efforts which are now being made through the introduction of "soft stone eliminators" and other devices to reduce these deleterious materials to a minimum. No satisfactory tests have ever been developed for determining the percentage of soft fragments in gravel. For the most part determinations are still made by inspectors on the basis of visual inspection, cutting with a knife blade, etc. A satisfactory test for soft fragments is urgently needed.

Tests for quality of fine aggregate are confined as a rule to strength and soundness. The old strength ratio test was supposed to be a test for quality. However, the marked effect of grading on the result led frquently to erroneous interpretations as to the significance of the test. The so-called "structural strength test" now stand-

ardized by the American Association of State Highway Officials and the American Society for Testing Materials theoretically takes care of the effect of grading, and to this extent is a great improvement over the old test. However, it still leaves much to be desired. So far as soundness tests on sand are concerned, about the same situation exists as for coarse aggregates.

Sand and Gravel in Bituminous Construction—This subject was also discussed in the paper.

What of the Future?—Until quite recently, studies of aggregate were confined largely to the effect of variations in grading and void content on the quality of mixtures—both portland cement and bituminous. However, it has become more and more evident that certain observed differences in the behavior of mixtures can only be accounted for by differences in aggregate characteristics other than grading. The mineral composition of the material, its thermal characteristics, its tendency to change in volume with change in moisture content, its surface texture and, in the case of bituminous mixtures, its angularity, are all being recognized as highly important in influencing the characteristics of the mixture. It is undoubtedly along these lines that research of the future will be directed. When we survey the almost limitless field which is opened up by these considerations we indeed realize how little we really know, even after all these years, regarding the physical characteristics of aggregates and their effect on the performance of road mixtures of all types.

THE AMERICAN CONCRETE INSTITUTE

SINCE its organization in 1905 the AMERICAN CONCRETE INSTITUTE has been devoted to the solution of technical problems arising from the use of cement in concrete and reinforced concrete and to the dissemination of information in this field of interest. It has sought to organize the needs and the curiosity of its members and the cooperation of many agencies outside that membership, to promote studies of the properties of concrete and of reinforced concrete and of their constituent and auxiliary materials, for the improvement of concrete design, construction and manufacture.

Annually since 1905 the Institute has held a convention; annually it has issued a volume of *Proceedings* and in these maintained a forum for the presentation and discussion of significant developments in the theory and practice of its field.

Since 1929 this work has been further facilitated by the publication of the JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, monthly except July and August*, affording more frequent contact with members and with the engineering profession generally.

The JOURNAL contents include:

Papers, reports and discussions originating in its own technical program.

Current Reviews, prepared by members, of the more outstanding and significant contributions to the literature of concrete, as these become available in the world's books, periodicals and engineering bulletins.

The Job Problems and Practice Department, inaugurated as a JOURNAL feature in 1938, provides place for an exchange of experience and of ideas in brief contributions from members.

News Letter pages presenting the news of the Institute and of developments and events tending to stimulate Institute work.

The work of the Institute is maintained by the dues paid by members and revenue from the additional sale of its publications. The scope of that work is evident in the list of contents of the annual volumes of *Proceedings*. The character of the work must be judged by a study of the work itself and by the personnel which has done

*This schedule was reduced so that: JOURNAL Vol. 4 (*Proceedings* Vol. 29) consists of 8 issues; JOURNAL Vol. 5, 6, 7, 8 and 9 (*Proceedings* Vol. 30, 31, 32, 33 and 34) is each complete in 5 issues. JOURNAL Vol. 10 (*Proceedings* Vol. 35) had a schedule of six issues. Vol. 11 has a similar schedule with issues for September and November, 1939, and January, February, April and June, 1940.

that work, for, very naturally the Institute's aims and its disinterested search for the guidance to be found in authoritative data have attracted a membership and a leadership which have constantly strengthened and furthered those aims.

Since A. C. I membership is not professionally or industrially homogeneous, its common ground of interest is in dependable basic knowledge of materials and of good practice in their use. It is not concerned with industry in an industrial sense. It's program is therefore free of the tendencies toward biased domination which often spring from a one-group viewpoint. The membership is recruited from among all those engineers, architects, contractors, manufacturers, research workers, chemists, physicists, and students who find the stimulus of adventure in solving the problems of a material which itself has solved more and more problems of engineering and architechtural economy and about which there is still so much to learn.

Annual dues (there is no initiation fee) in the various member classes, are:

Members:	
Individual	$10.00
Firm, Corporation, Society or Other Organization	20.00
Contributing Members	50.00
Student Members	4.00

Life Members (see Article IV, Sec. 1, By-Laws).
Dues include JOURNAL subscription.
Non-Members may subscribe to the JOURNAL at $7.50 per year.

The American Concrete Institute provides a comradeship in finding the best ways to do concrete work of all kinds and in spreading that knowledge. Further information may be had by addressing the Secretary, AMERICAN CONCRETE INSTITUTE, 7400 Second Boulevard, Detroit, Michigan.

Charter

(The original charter as it appears below was granted to The National Association of Cement Users. An amended Certificate of Incorporation, authorized by the Board of Direction May 29, 1913, drawn by the President and Secretary July 1, 1913, recorded July 2, 1913 in the office of the Register of Deeds, of the District of Columbia, changed the name of the organization to American Concrete Institute.—*Secretary*)

KNOW ALL MEN BY THESE PRESENTS, That we, the undersigned, all of whom are citizens of the United States, and a majority of whom are residents of the District of Columbia, have associated ourselves together for the purpose hereinafter set forth and desiring that we may be incorporated as an Association under sub-chapter three (3) of the Incorporation Laws of the District of Columbia, as provided in the Code of Law of the District of Columbia, enacted by Congress and approved by the President of the United States, do hereby certify:

1. Name. The name of the proposed corporation is "The National Association of Cement Users."

2. Term of Existence. The existence of the said corporation shall be perpetual.

3. Objects. The particular business and objects of the said corporation shall be to disseminate information and experience upon and to promote the best methods to be employed in the various uses of cement by means of convention, the reading and discussion of papers upon materials of a cement nature and their uses, by social and friendly intercourse at such conventions, the exhibition and study of materials, machinery and methods and to circulate among its members by means of publications the information thus obtained.

4. Incorporators. The number of its managers for the first year shall be fifteen.

In Witness Whereof. we have hereunto set our hands and seals this fourteenth day of December, A. D., 1906.

RICHARD L. HUMPHREY, (SEAL)
JOHN STEPHEN SEWELL, (SEAL)
S. S. VOORHEES. (SEAL)

OFFICE OF RECORDER OF DEEDS,
DISTRICT OF COLUMBIA.

This is to certify that the foregoing is a true and verified copy of a Certificate of Incorporation, and of the whole of such Certificate as received for record in this office at 9:49 A. M., the 19th day of December, A. D., 1906.

In testimony whereof I have hereunto set my hand and affixed the seal of this office, this 20th day of December, A. D., 1906.

(Signed) R. W. DUTTON,
Deputy Recorder of Deeds,
District of Columbia.

(SEAL)

By-Laws*

ARTICLE I—MEMBERS

Section 1. This Institute shall consist of Members, Contributing Members, Student Members and Honorary Members interested in furthering the Institute's objects as set forth in its Charter.

Sec. 2. A Member or a Contributing Member may be a person, firm, corporation, society or other organization. A Student Member shall be a student in residence at a recognized technical or engineering school. An Honorary Member shall be a person of eminence in the field of the Institute's interest, or one who has performed extraordinarily meritorious service to the Institute.

Sec. 3. A Member or Contributing Member shall be proposed by two Members of the Institute and elected by a two-thirds vote of the Board of Direction. A Student Member shall be proposed by a member of the faculty of his institution and elected by a two-thirds vote of the Board of Direction. An Honorary Member shall be nominated by ten members of the Institute and elected by unanimous vote of the Board of Direction in secret ballot.

Sec. 4. All members shall have all rights and privileges of membership as determined by the Board of Direction except that a Student Member shall not vote nor hold office. The status of a Student Member shall change automatically to that of Member on the first anniversary of his membership succeeding the date on which he ceases to be a student in residence.

Sec. 5. A firm, corporation, society or other organization holding membership may name one individual as its representative who shall enjoy all membership rights and privileges.

Sec. 6. Applications for membership on forms prescribed by the Board of Direction, and resignations from membership, shall be presented in writing to the Secretary-Treasurer. Resignations may be accepted only from members whose dues are not more than sixty days in arrears, except by special action of the Board of Direction.

ARTICLE II—OFFICERS

Section 1. The officers shall be a President, two Vice-Presidents, six Regional Directors (one from each of six geographical districts), three Directors-at-large, and the Secretary-Treasurer, who with the five latest, living Past Presidents who continue to be members, shall consititute the Board of Direction.

Sec. 2. The Board of Direction shall from time to time divide the territory occupied by the membership in the six geographical districts to be designated by numbers, each to be represented by a Regional Director, as provided in Section 1.

Sec. 3. The President, Vice-Presidents, Directors-at-large, Regional Directors, and five members of a Committee on Nominations shall be elected by letter ballot of the Institute membership. The Secretary-Treasurer shall be appointed annually by the Board of Direction.

Sec. 4. Before September 15 of each year the Committee on Nominations shall, by letter ballot of its members, nominate candidates for offices to become vacant

*As amended by the 33rd Annual Convention February 1937 and ratified by letter ballot canvassed May 25, 1937.

at the next annual convention and twenty candidates for membership on the Committee on Nominations which is to serve in the following year and shall transmit the names of all candidates thus nominated to the Secretary-Treasurer of the Institute. The consent of each candidate for office must be obtained before notice of his nomination is published. The Secretary-Treasurer shall cause notice of all such nominations to be transmitted to the membership of the Institute at least 120 days prior to the next ensuing annual convention. Upon petition to the Board of Direction signed by at least ten members of the Institute, additional nominations for offices or for membership on the Committee on Nominations may be made within 30 days thereafter.

The complete list of nominations shall be submitted 60 days before the next convention to the Institute membership for letter ballot to be canvassed at 5 p. m. on the first day of the convention and the result announced at a session of the convention on the second day. The candidate for any office receiving the most votes shall be declared elected and the candidate receiving the most votes for membership on the Committee on Nominations shall be Chairman of that committee; the four next highest shall be declared elected members of the Committee. With these five the three latest past president members of the Board of Direction shall serve, making a total membership of eight.

Should any member of the Committee on Nominations thus chosen fail, within fifteen days of formal notice from the Secretary-Treasurer, to make written acceptance of service, a vacancy shall occur to be filled by the candidate receiving the next greatest number of votes and so on until the five elective places on the committee shall be filled.

Sec. 5. The terms of office of the President, Vice-Presidents, Secretary-Treasurer and Regional Directors shall be one year. Directors-at-large shall be elected one each year for a term of three years. A year is to be here construed as the period between adjournments of two successive annual conventions.

Sec. 6. The President, Vice-Presidents and Regional Directors shall be ineligible for more than one re-election to the same office until the lapse of at least one term.

Sec. 7. The term of each officer shall begin at the close of the annual convention at which he is elected and shall continue until a successor is duly elected.

Sec. 8. A vacancy in the office of President shall be filled by the Vice-President having Institute membership seniority.

Sec. 9. Vacancy in any office, for the unexpired term, shall be filled by appointment by the Board of Direction except as provided in Section 8.

Sec. 10. In the event of disability or neglect in the performance of his duty of any officer of the Institute, the Board of Direction shall have the power to declare the office vacant.

Sec. 11. The Board of Direction shall have general supervision of the affairs of the Institute. At a meeting held in the week of the annual convention it shall appoint a Secretary-Treasurer for a term of one year. It shall authorize and appoint the chairmen of such administrative and technical committees and assign to them such duties and such authority as it deems needful to carry on the work of the Institute. Additional committee members shall be appointed by the President.

Sec. 12. There shall be an Executive Committee of the Board of Direction consisting of the President, Secretary-Treasurer, and three of its members appointed by the Board of Direction.

Sec. 13. The Executive Committee shall manage the affairs of the Institute during the interim between the meetings of the Board of Direction.

Sec. 14. The President shall perform the usual duties of the office. He shall preside at the annual convention, at the meetings of the Board of Direction and of the Executive Committee, and shall be ex-officio member of all committees. He may name a chairman to serve in his place for any sessions of the convention.

The Vice-Presidents, each in the order of his Institute membership seniority, shall discharge the duties of the President in his absence. In the absence of President and both Vice-Presidents a President Pro-Tem, appointed by the Board, shall discharge such duties.

Sec. 15. The Secretary-Treasurer shall perform such duties, furnish such bond and receive such salary as shall be determined by the Board of Direction.

ARTICLE III—MEETINGS

Section 1. The Institute shall hold an annual convention and such other meetings as may be authorized by the Board of Direction. The time and place of all meetings shall be fixed by the Board of Direction. Notice of this action shall be sent to all members at least thirty days previous to the date of each meeting.

Sec. 2. The Board of Direction shall meet at least twice each year, the time and place fixed by the Board.

Sec. 3. The Executive Committee shall meet on call of the President or of any three of its members.

Sec. 4. Twenty-five members shall constitute a quorum for meetings of the Institute; eight members shall constitute a quorum for meetings of the Board of Direction; and three members for meetings of the Executive Committee.

ARTICLE IV—DUES

Section 1. Dues for the several membership classes shall be payable annually in advance from the first of the month of notification of the member applicant of his election by the Board of Direction as follows: Contributing members, $50.00; Members, individuals, $10.00; firms, corporations, societies or other organizations, $20.00; Student members, $4.00; Honorary members, none. Any individual member may be admitted to life membership upon payment of a sum determined by the Executive committee based on 90 per cent of the membership dues as established at the time of application, credited with 3 per cent interest compounded annually for the applicant's life expectancy as arrived at from the American Experience Table of Mortality.

Sec. 2. A member shall be entitled to receive one copy of each issue of the JOURNAL of the AMERICAN CONCRETE INSTITUTE as issued in the period of his membership and additional or other publications as determined by the Board of Direction.

Sec. 3. A member whose dues remain unpaid for a period of six months shall forfeit the privilege of membership and shall be officially notified to this effect by the Secretary-Treasurer. If these dues are not paid within six months thereafter his name shall be stricken from the list of members, unless otherwise specifically ordered by the Board of Direction. Members may be reinstated upon payment of all indebtedness against them upon the books of the Institute.

ARTICLE V—STANDARDS

Section 1. The Board of Direction shall adopt such rules and regulations governing the procedure for the consideration and adoption of Standard Specifications,

Standard Practice Recommendations and Standard Definitions as will safe-guard their soundness and representative character.

ARTICLE VI—AMENDMENTS

Section 1. Proposed amendments to these By-Laws, signed by at least fifteen members, if presented in writing to the Board of Direction ninety days before the annual convention, shall be mailed to the membership at least thirty days prior to the annual convention. These amendments may be discussed and amended at the annual convention and be passed to letter ballot by a two-thirds vote of those present. Two-thirds of the votes cast by letter-ballot canvassed within ninety days thereafter shall be necessary for their adoption.

Officers and Directors

(Officers and Regional Directors are elected by letter ballot canvassed at each annual convention for terms of one year—Directors-at-large, one each year for a term of three years.)

President

F. E. Richart

Vice-Presidents

R. B. Young — Ben Moreell

Secretary-Treasurer

Harvey Whipple

Directors:

1st District—John R. Nichols
2nd District—F. E. Schmitt
3rd District—Lion Gardiner
4th District—R. W. Crum
5th District—A. S. Douglass
6th District—H. F. Thomson

Directors-at-large

Benjamin F. Affleck (Term expires February 1940)
Raymond E. Davis (Term expires February 1941)
M. O. Withey (Term expires February 1942)

Past Presidents

Richard L. Humphrey (1905-14) Deceased
Leonard C. Wason (1915-16) Deceased
William K. Hatt (1917-19)
Henry C. Turner (1920-21)
William P. Anderson (1922-23)
A. E. Lindau (1924-25)
M. M. Upson (1926-27)
E. D. Boyer (1928-29) Deceased
D. A. Abrams (1930-31)
S. C. Hollister (1932-33)
*Arthur R. Lord (Feb.-July, 1934)
*P. H. Bates (July 1934-Feb. 1936)
*F. R. McMillan (1936)
*J. C. Pearson (1937)
*John J. Earley (1938)

*The five latest living past presidents are members of the Board.

Executive Committee

F. E. Richart — J. C. Pearson
John J. Earley — R. B. Young
Harvey Whipple

Conduct of Committee Work

Adopted by the Board of Direction February 21, 1938 under the authority of the By-Laws, Article II, section 11, as follows: "The Board of Direction shall have general supervision of the affairs of the Institute. At a meeting held in the week of the annual convention it shall appoint a Secretary-Treasurer for a term of one year. It shall authorize and appoint the chairmen of such administrative and technical committees and assign to them such duties and such authority as it deems needful to carry on the work of the Institute. Additional committee members shall be appointed by the President".

INTENT

It is the intent here to define the scope of and the responsibility for work assigned to administrative and to technical committees of the Institute and to leave to each such working group as full a measure of freedom for the exercise of initiative as the necessities of orderly organization procedure permit.

ADMINISTRATIVE COMMITTEES—GENERAL

(1) Administrative committees are considered arms of the Board of Direction assigned to administer Board policy in their respective divisions of Institute activity. Each such committee shall make a report of its work to the Board at least once each year or at such other more frequent intervals as the Board may from time to time require. The Secretary-Treasurer of the Institute shall be the secretary of each such committee.

(2) Before putting into effect any significant or material changes from the procedure of recent years, and before making engagements or commitments contingent upon the approval of such plans administrative committees shall secure Board approval.

(3) The chairman and secretary of each administrative committee shall be the executive group of the committee between meetings and for all work not specifically assigned to some other member or members, except as hereinafter provided.

ADVISORY COMMITTEE

(1) The duties and responsibilities of the Advisory Committee shall be those which arise in effecting productive technical committee activity. It shall initiate, plan and co-ordinate technical committee assignments in relation to the developments and the needs of the times and the competent personnel available for specific tasks. It shall endeavor to further committee undertakings and keep informed of progress on specific assignments, cooperating with other administrative committees and affiliated organizations. It shall make recommendations to the Board of Direction for the creation of new and the reorganization, reassignment or discharge of existing committees. It shall nominate committee personnel and define specific committee assignments.

(2) The Advisory Committee shall assign each new technical committee organized to that one of the nine committee departments (hereinafter listed) with whose field of interest its task is most clearly identified and designate the committee by title and a number in its departmental series.

(3) The Advisory Committee shall be composed of a chairman, a secretary (the Secretary-Treasurer of the Institute) the chairmen of Program, Publications, and Standards committees and nine other members, to represent the viewpoint and be chairman of one of the following nine departments of committee activity:

Departments

100—*Research*—To comprise all committee work in which the principal interest is in the interpretation of test data or the initiation or direction of tests.

200—*Materials*—To comprise all committee work in which the principal interest is in the production and distribution of materials used in concrete work.

300—*Engineering design*—To comprise all committee work in which the principal interest is in structural design or in the supervision of construction of such work.

400—*Architectural design*—To comprise all committee work in which the principal interest is in architectural design or in the supervision of such work.

500—*Specifications*—To comprise all committee work in which the principal interest is in the production of specifications for material, services or workmanship on behalf of the owner.

600—*Field construction*—To comprise all committee work in which the principal interest is monolithic construction, including job equipment, organization, and methods.

700—*Shop manufacture*—To comprise all committee work in which the principal interest is in the manufacture and distribution of factory-made concrete products.

800—*Special requirements*—To comprise all committee work in which the principal interest arises from the requirement of unusual or special qualities in concrete on account of its use in less common construction or where subject to extraordinary exposures.

900—*Joint efforts*—To comprise all committee work done by the Institute jointly with representatives from other societies and work in which the principal interest is clearly in two or more of the foregoing departments.

(4) The chairman and secretary of the Advisory Committee shall be members, ex-officio, of all technical committees and shall receive notice of meetings, minutes, drafts of reports, etc., the same as other technical committee members. The chairman of each department shall be a member ex-officio of each committee of his department and shall receive notice of meetings, minutes, drafts of reports, etc., the same as other technical committee members.

PROGRAM COMMITTEE

(1) The Program Committee shall have general charge of the programs of all Institute conventions and general meetings and shall prescribe their subject matter. For this purpose it may have first choice of papers and reports submitted to the Institute and determine whether contributions so selected shall be published (if in the judgment of the Publications Committee they are acceptable for publication) in advance of or following the convention at which they are to be presented. The Program Committee may draw upon papers and reports published in the JOURNAL or invite for convention use papers not previously offered for publication.

(2) The Program Committee shall be composed of a chairman, a secretary (the Secretary-Treasurer of the Institute) the chairmen of Advisory, Publications and Standards committees and not more than four other members.

PUBLICATIONS COMMITTEE

(1) The Publications Committee shall have general charge of the JOURNAL OF THE AMERICAN CONCRETE INSTITUTE and of other Institute technical publications and shall determine their scope, character and content in conformity with Board aims and policies. It shall invite papers for publication, determine their acceptability and supervise their editorial treatment. This committee shall be composed of a chairman, a secretary and editor (the Secretary-Treasurer of the Institute), chairmen of Advisory, Program, and Standards committees and not more than four other members.

MEDALS AWARD COMMITTEE

(1) This committee shall recommend to the Board the persons who shall receive the Wason Medals, the Turner Medals and any other medals or awards which may be established, in accordance with the conditions set up by the donors or by the Board. The membership of this committee shall not be publicly announced except by special vote of the Board. The Secretary-Treasurer of the Institute as secretary of this committee shall have no vote.

STANDARDS COMMITTEE

(1) The Standards Committee shall have supervision of the promulgation of Standards and proposed standards of the Institute under the provisions of rules (adopted by the Board Feb. 22, 1937, under the authority of the By-Laws, Article V), as follows:

(a) Subject to its responsibility under the By-Laws for Standards of the Institute the Board delegates jointly to the Advisory and Standards committees the administration of the Institute's work in the promulgation of Standards. With the Advisory committee is the responsibility for the initiation of technical committee standardizing efforts and the nomination of committee personnel in each instance, with due regard for the importance of such a balanced and thoroughly representative committee membership as will, at the outset, tend to safeguard the "soundness and representative character" of the standards to be written.

(b) When a technical committee for the development of a standard has been organized, notice of its assignment and personnel shall be published in the JOURNAL and it shall be the responsibility of the Secretary-Treasurer of the Institute to bring such notice to the attention of societies known to have an interest in, or be in a position to submit data pertinent to the work to be undertaken.

(c) When a new or a revised standard is proposed by a majority in a technical committee assigned by the Board to its preparation and upon its receipt by the Secretary-Treasurer of the Institute, it shall be referred to the Standards Committee, which shall be responsible for its critical examination to determine its worthiness as a document sufficiently comprehensive and representative in character to be submitted to the Institute with a view to adoption as a Standard of the Institute.

(d) The Standards Committee shall have a Chairman and six other members, appointed for such terms that two new or re-appointments shall be made each year. The Secretary of the Institute shall be an additional, non-voting member, and shall be the secretary of the committee. The membership of the Standards Committee shall be further augmented for the sole purpose of reviewing a proposed standard, by the chairman (or member designated by the chairman) of the committee proposing the Standard for adoption but he shall not vote as a member of the Standards Committee.

(e) So constituted the Standards Committee shall adopt such means—whether by correspondence, hearings, the appointment of a sub-committee or by limited distributions of a proposed standard to selected individuals or agencies—as it deems expedient to satisfy itself of the adequacy (in view of the purposes cited above) of the proposed standard. If and when approved by at least four members of the Standards Committee it shall be transmitted, through the Secretary-Treasurer of the Institute to the Publications Committee with recommendations for a prescribed preliminary distribution as a basis for criticism and discussion prior to its formal presentation to the Institute with a view to adoption.

(f) The nature and extent of this preliminary distribution and the interval prior to formal presentation to the Institute shall be determined by the Standards Committee after consultation with the technical committee and the Publications Committee.

(g) In the event that four or more members of the Standards Committee do not approve the proposed standard it shall be referred back to the technical originating committee with a statement of the reason or reasons for rejection.

(h) A proposed standard may be re-submitted to the Standards Committee when accompanied by a statement from the originating committee setting forth the procedure in the reconsideration of the proposed standard since first submitted.

(i) In the event of deadlock or of apparent obstruction in the outlined procedure, the Advisory Committee, the Standards Committee or the technical committee originating the proposed standard under consideration may appeal to the Board of Direction.

(j) When a standard is formally presented to the Institute with the approval of four or more members of the Standards Committee, it shall be open for discussion on the floor of a convention. If approved by a majority vote of members present it shall be submitted, by reference, to letter ballot of the Institute membership and the standard shall become effective upon ratification by two thirds of those voting. An amendment to a proposed standard presented on the floor of the convention shall be sustained only when approved by nine tenths of the members voting and as so amended shall be submitted to letter ballot as provided above.

*(k) The existing standards of the American Concrete Institute shall be the special responsibility of the Advisory Committee which at intervals of not to exceed four years shall constitute (or reconstitute as the case may be) a committee to reconsider each of the existing standards and to report it to the Institute with recommendations for adoption, revised or unrevised.

TECHNICAL COMMITTEE MEMBERSHIP, ORGANIZATION and PROCEDURE

(1) Committees shall be composed of individuals nominated by the Advisory Committee; the chairmen appointed by the Board and other members by the President. They shall be selected not only for their knowledge of, experience and judgment in the field of each committee's assignment, but also in reference to their situation as it may contribute to their active attention to the committee's work. In general, since the policy of the Institute is to organize many of its committees for and assign them to specific tasks, their membership may be limited at the outset to relatively small functional groups with a view to subsequent appeal to competent critics of the reports submitted—either as additional members of the committee or in a capacity assisting the Publications Committee.

*Rules adopted Feb. 22, 1937 were revised Feb. 28, 1939, in this paragraph (k) as it now appears replacing the former provision with respect to expiration of standards, which was as follows: "A Standard of the Institute which is unrevised for three years shall expire automatically unless re-adopted."

(2) Each technical committee authorized by the Board of Direction shall receive from the Advisory Committee of the Institute an explicit statement of its assignment.

(3) A committee when organized shall plan its own procedure. Its chairman or its secretary shall advise all members of the committee in writing what procedure is to be followed in the assigned undertaking. Each committee member is to be advised subsequently by chairman or secretary as the work progresses. Copies of all such correspondence and all notices and minutes of meetings are to be sent to the chairman and the secretary of the Advisory Committee, and to the chairman of the department with which the technical committee's work is identified.

(4) Committee meetings are to be encouraged but since frequent meetings are often a hardship on committee members, the Board recognizes that much effective work can be done by correspondence by the procedure of trial and criticism wherein one or more members undertake, after exchanges of views, the preliminary drafts of a report.

(5) Committees shall serve for the terms for which they are appointed, or in the absence of a prescribed term of service, for one year from the end of the fiscal year (June 30) succeeding the JOURNAL announcement of the committee's organization. Except as continued with the same, a new or revised assignment and specifically with the same or reorganization in membership, committees are automatically discharged at the end of the prescribed period. While this is in keeping with a policy in general to assign committees to definite tasks rather than to fields of interest, it is not intended to discourage the desirable practice of technical committees in recommending further work.

(6) In submitting a report on its assignment, a technical committee shall present a supplementary report for the Advisory Committee with recommendations of further studies and additions to or changes in personnel suggested by the work done on its specific assignment.

(7) Accompanying its report as submitted to the Secretary-Treasurer of the Institute the chairman or secretary of the technical committee shall submit solely for the information of Advisory and Publications committees, (a) a statement of anything significant in the committee's activity with reference to individual response and performance and sources of important data, and (b) to appear with the report if and when published, a further statement in either of the following forms:

(a) This report was submitted to letter ballot of the committee which consists of —— members; —— members returned their ballots of whom —— have voted affirmatively and —— negatively.

(b) This report was approved in form and substance as here submitted by the following members of the Committee (followed by names of the approving members).

(8) No committee report or part thereof shall be released for publication except through the Publications Committee of the Institute.

TECHNICAL COMMITTEE EXPENSES AND SPECIAL FUNDS

(1) The Secretary-Treasurer will honor vouchers from a committee chairman for postage expense incurred in the work of his committee, and will supply committee stationery on request. Other committee expenses will not be assumed by the Institute unless previously authorized specifically by the Board.

(2) Special committee funds, when their solicitation has been authorized by the Board, shall be deposited to the Institute's account through the Institute's Secretary-Treasurer and be disbursed on vouchers signed by the Chairman of the Committee for whose work the fund was created.

DISTRIBUTION OF TECHNICAL COMMITTEE REPORTS

(1) Except as provided with reference to proposed Standards (see "Standards Committee") the distribution of committee reports shall be in the hands of the Publications Committee. In general a committee report will have a preliminary, very limited, distribution to critics named by the Publications Committee and subsequent publication complete or in part, as the Publications Committee shall direct.

THE JOURNAL AND THE CONVENTION IN RELATION TO TECHNICAL COMMITTEE WORK

The work of the Institute is continuous throughout the year. Conventions and JOURNAL issues present a series of forums. One of the important considerations in inaugurating the publication of the *Proceedings* periodically in the JOURNAL was to increase the scope and volume of Institute work, to give consideration to more papers and committee reports than can be considered adequately in the limited periods of a convention. Thus technical committees will not assume that a report made available to the Institute is necessarily for convention presentation. The pre-convention and post-convention periods are over-crowded. Committees are urged to complete their work in early spring or late summer. It is the policy of the Institute through the Program Committee to relieve the strain of over-crowded convention sessions. Committees should submit their reports as promptly as possible to the Secretary-Treasurer of the Institute to be considered by the Publications Committee, or if a proposed Standard is involved, by the Standards Committee. The interlocking membership of Advisory, Program, Publications and Standards Committees through their respective chairmen serves to keep all of them advised of the completion of a committee task. The Program Committee usually begins its work on a convention program before all the papers and reports of the previous convention have been published. It is in a position, if a committee report is available early enough and if it seems to present subject matter which could be better considered through discussion in the meeting room than by correspondence, to schedule it for convention presentation. It is impracticable and is likely to become more and more impossible for Program Committees to put all available reports and papers on a convention program.

Administrative Committees

Advisory

The Advisory Committee's responsibility is in recommendations to the Board of Direction as to technical committee undertakings.*

RAYMOND E. DAVIS, Chairman
HARVEY WHIPPLE, Secretary
M. O. WITHEY, Chairman Department 100, Research
STANTON WALKER, Chairman Department 200, Materials
A. W. STEPHENS, Chairman Department 300, Engineering Design
A. J. BOASE, Chairman Department 400, Architectural Design
MILES N. CLAIR, Chairman Department 500, Specifications
R. B. YOUNG, Chairman Department 600, Field Construction
W. G. KAISER, Chairman Department 700, Shop Manufacture
RAYMOND E. DAVIS, Chairman Department 800, Special Requirements
F. H. JACKSON, Chairman Department 900, Joint Efforts
P. H. BATES, Chairman Publications Committee
R. W. CRUM, Chairman Standards Committee
F. R. MCMILLAN, Chairman Program Committee

Program

The Program Committee's work is to select from among papers and reports submitted to the Institute such contributions as it deems desirable for the annual conventions.*

F. R. MCMILLAN, Chairman
HARVEY WHIPPLE, Secretary
F. H. JACKSON
A. E. LINDAU
BEN MOREELL
D. E. PARSONS
F. E. SCHMITT
R. B. YOUNG
P. H. BATES, Chairman Publications Committee
R. W. CRUM, Chairman Standards Committee
RAYMOND E. DAVIS, Chairman Advisory Committee

Publications

The Publications Committee has the responsibility of the technical publications of the Institute and invites original papers furthering the objects of the society and discussions of published papers and reports. It passes upon the papers and the

*See "Conduct of Committee Work," p. 617)

reports of committees submitted for publication. (See also Standards Committee). All contributions are received through the Secretary-Treasurer who is the editor of the JOURNAL and Secretary of the committee.*

P. H. BATES, Chairman
HARVEY WHIPPLE, Secretary
H. J. GILKEY
BENJAMIN WILK
R. B. YOUNG
R. R. ZIPPRODT
R. W. CRUM, Chairman Standards Committee
F. R. MCMILLAN, Chairman Program Committee
RAYMOND E. DAVIS, Chairman Advisory Committee

Standards

The Standards Committee has supervision of the promulgation of Standards and proposed Standards of the Institute under the provisions of Board rules on "Standards."*

R. W. CRUM, Chairman
HARVEY WHIPPLE, Secretary
P. H. BATES, (to June 30, 1941)
R. L. BERTIN (to June 30, 1941)
MILES N. CLAIR (to June 30, 1941)
W. G. KAISER (to June 30, 1941)
C. T. MORRIS (to June 30, 1940)
STANTON WALKER (to June 30, 1940)

*See "Conduct of Committee Work," p. 617)

Technical Committees

Following is a list of Institute Technical Committees for the year beginning July 1, 1939.

Institute Committees are not "standing" committees; do not continue automatically from year to year. They are, in general, assigned for a specified term to immediate tasks rather than to fields of interest. These assignments may be and frequently are continued from year to year.

Department 100—Research

M. O. WITHEY, Chairman

108—Properties of Mass Concrete—RAYMOND E. DAVIS—Chairman
(Dept. of Civil Engineering, University of Calif., Berkeley)

F. R. MCMILLAN—Vice-chairman

P. H. BATES	W. C. HANNA	J. L. SAVAGE
R. F. BLANKS	J. W. KELLY	B. W. STEELE
R. W. CARLSON	S. B. MORRIS	I. L. TYLER
J. P. GROWDON		C. P. WILLIAMS

Papers presented at the 34th annual convention by four members of this committee plus introductory remarks by Chairman Davis (see March-April and May-June, 1938 JOURNALS) serve as a propectus of the report which the committee expects to have available for publication in the near future—summarizing and presenting conclusions on the vast research on the properties of mass concrete.

113—Bond Stress—VERNON P. JENSEN—Chairman
(University of Illinois, Urbana)

G. C. ERNST	R. E. GLOVER	F. E. RICHART
H. J. GILKEY	G. A. MANEY	J. R. SHANK
H. F. GONNERMAN	WARREN RAEDER	ALBERT SMITH

This committee is assigned the task of promoting research and collecting and interpreting data from field and laboratory with a view to determining proper working stresses for bond in reinforced concrete construction.

115—Research—M. O. WITHEY*—Chairman
(Professor of Mechanics, University of Wisconsin, Madison)

F. E. RICHART*—Vice-Chairman
(Research Professor of Engineering Materials, University of Illinois, Urbana)

R. W. BEAL—Secretary
(Iowa State College, Ames, Ia.)

TRACY BARTHOLOMEW
(Duquesne Slag Products Co.)
E. F. BERRY
(Syracuse University)
J. A. C. CALLAN
(Alabama Polytechnic Institute)
R. W. CARLSON
(Massachusetts Institute of Technology)
M. N. CLAIR
(Thompson & Lichtner Co., Inc.)
R. B. CREPPS
(Purdue University)
R. E. DAVIS*
(University of California)
N. H. DOUGHERTY
(Tennessee University)
H. H. EDWARDS
(Associated Colleges of Claremont)
GEORGE C. ERNST
(University of Maryland)
R. D. FINNEY
(Kansas State Highway Dept.)
P. J. FREEMAN
(Tennessee Valley Authority)
W. E. GIBSON
(Kansas State Highway Com.)
F. E. GIESECKE
(Texas A. & M. College)
H. J. GILKEY
(Iowa State College)
A. G. GILLESPIE
(Rock Island Arsenal)
V. L. GLOVER
(Illinois State Highway Dept.)
A. T. GOLDBECK
(National Crushed Stone Assn.)
H. F. GONNERMAN*
(Portland Cement Association)
S. H. GRAF
(Oregon State Agricultural College)
L. E. GRINTER
(Armour Institute of Technology)
S. O. HARPER
(Bureau of Reclamation)
W. T. HARTMAN
(Virginia Polytechnic Institute)
S. C. HOLLISTER
(Cornell University)
C. A. HUGHES
(University of Minnesota)
ROBERT HUMPHREYS
(Kentucky State Highway Dept.)
A. S. JANSSEN
(University of Idaho)
BRUCE JOHNSTON
(Lehigh University)
H. L. KENNEDY
(Dewey & Almy Chemical Co.)
WILLIAM J. KREFELD
(Columbia University)
G. H. LARSON
(Wisconsin State Highway Dept.)
T. R. LAWSON
(Rensseler Polytechnic Institute)
H. W. LEAVITT
(University of Maine)
B. G. LONG
(Cincinnati Testing Laboratory, U. S. A.)
H. J. LOVE
(National Slag Association)
T. H. MACDONALD
(Bureau of Public Roads)
C. E. MAGNUSSON
(University of Washington)
G. A. MANEY
(Northwestern University)
JOSEPH MARIN
(Rutgers University)
SEWARD MASON
(Montana State Highway Dept.)
H. S. MATTIMORE
(Pennsylvania State Highway Dept.)
JAMES MCCORMICK
(New Hampshire State Highway Dept.)
W. W. MCLAUGHLIN
(Michigan State Highway Dept.)
W. C. MCNOWN
(University of Idaho)
BERT MYERS
(Iowa State Highway Com.)
W. H. MILLS, JR.
(So. Carolina State Highway Dept.)
CLYDE T. MORRIS*
(Ohio State University)
S. B. MORRIS
(Stanford University)
LEVI MUIR
(Utah State Highway Dept.)
J. M. MURDOUGH
(Texas Technological College)
T. D. MYLREA
(University of Delaware)
O. NEIL OLSON
(Marquette University)
D. E. PARSONS*
(National Bureau of Standards)
J. C. PEARSON*
(Lehigh Portland Cement Co.)
J. S. PECK
(College of the City of New York)
C. J. POSEY
(University of Iowa)
E. W. RAEDER
(University of Colorado)
P. L. REED
(University of Florida)
CARL R. REID
(Oaklahoma State Highway Dept.
F. E. RICHART*
(University of Illinois)
E. H. ROCKWELL
(Lafayette College)
R. I. ROWELL
(Vermont State Highway Dept.)

*Members of the Committee's Executive Group.

R. D. SCHEIER
(Pennsylvania State College)
E. W. SCRIPTURE, JR.
(Master Builders Research Laboratory)
J. R. SHANK
(Ohio State University)
R. H. SHERLOCK
(University of Michigan)
W. Y. SKILES
(Georgia School of Technology)
T. E. STANTON
(California State Highway Dept.)
M. A. SWAYZE
(Lone Star Cement Corp.)
E. O. SWEETSER
(Washington University)
BAILEY TREMPER
(Washington State Highway Dept.)
HARRY TUCKER
(University of North Carolina)
E. VIENS
(Ottawa Laboratory of Testing Materials)
STANTON WALKER
(National Sand and Gravel Assn.)
C. E. WILLIAMS
(Battelle Memorial Institute)
T. F. WILLIS
(Missouri State Highway Dept.)
M. O. WITHEY*
(University of Wisconsin)
C. E. WUERPEL
(No. Atlantic Division War Dept.)
R. B. YOUNG
(Hydro Electric Power Comm. of Ontario)
R. R. ZIPPRODT
(Columbia University)

*Members of the Committee's Executive Group.

This committee was organized to affiliate the interests of research workers and agencies in the field of concrete. Its assignment is to review and correlate research in concrete and reinforced concrete and to consider research methods and objectives. The open sessions of the committee at the 33rd, 34th and 35th annual conventions of the Institute have attracted considerable and an apparently very much interested attendance. Because the Institute has agreed not to publish the transcript of these meetings research men have felt free to discuss projects proposed and under way for the benefit of their fellows in the research field.

116—Volume Changes and Plastic Flow in Concrete—G. E. TROXELL—Chairman
(Engineering Materials Laboratory, University of California, Berkeley)

R. W. CARLSON, H. E. DAVIS, R. E. DAVIS, H. J. GILKEY, H. S. MEISSNER, F. E. RICHART, J. R. SHANK, G. E. TROXELL

This committee consolidates the previous assignment of Committee 102, "Volume Changes in Concrete" and Committee 109, "Plastic Flow." The subjects proved to be virtually inseparable and the conclusions of and the implications of research in these two subjects will be reported by the new committee.

Department 300, Engineering Design

A. W. STEPHENS, Chairman

312—Plain and Reinforced Concrete Arches—CHARLES S. WHITNEY—Chairman
(724 E. Mason St., Milwaukee, Wis.)

W. S. COTTINGHAM, CLYDE T. MORRIS, C. E. MORGAN, W. M. WILSON

This committee has been asked to prepare a specification for reinforced concrete arch design—giving effect to all that has been learned of the influence of dead and live loads and volume changes due to shrinkage, plastic flow and temperature change.

314—Rigid Frame Bridges—Committee being reorganized under chairmanship of D. H. PLETTA, Virginia Polytechnic Institute, Blacksburg, Va.

315—Detailing Continuous Beams and Frames—A. J. BOASE—Chairman
(Portland Cement Assn., 33 West Grand Ave., Chicago)

E. P. ALLABACH	BENGT FRIBERG	T. D. MYLREA
FRANK H. BEINHAUER	J. E. KALINKA	FRED L. PLUMMER
R. L. BERTIN	FRANK KEREKES	GEORGE P. RICE
E. E. EBLING	A. E. LINDAU	C. N. SCHWERTNER
HARRIS EPSTEIN	A. B. MACMILLAN	WILLIAM F. ZABRISKIE
	T. J. MCCORMICK	

This committee was asked to recommend improved practice to avoid cumbersome methods now in use in the delineation on the engineer's drawings of the results of the designer's analysis of reinforced concrete frame buildings—the eventual report to meet the need growing out of the fact that practice has not kept pace with the advance in design methods.

317—Reinforced Concrete Building Design Handbook—A. J. BOASE, Chairman

H. P. BIGLER	L. E. GRINTER	A. E. LINDAU
HARRY ELLSBERG	MOGENS IPSEN	F. E. RICHART

The committee is preparing a new handbook of reinforced concrete building design.

Department 400, Architectural Design

A. J. BOASE, Chairman

408—Recommended Practice for the Use of Color in Concrete—(This committee is being reorganized)

412—Specifications and Recommended Practice for Architectural Concrete of the Exposed Aggregate Type—J. C. PEARSON, Chairman
(Care Lehigh Portland Cement Co., Allentown, Pa.)

P. H. BATES	R. W. CARLSON	F. R. MCMILLAN
A. J. BOASE	JOHN J. EARLEY	R. H. SHREVE

This committee as its title implies, is assigned to a considerable task in writing specifications and recommended practice for architectural concrete of the exposed aggregate type, covering basic considerations of materials, mix design, manipulation, craftsmanship and manufacturing practice as well as structural design.

Department 500, Specifications

MILES N. CLAIR, Chairman

501—Standard Building Code—A. W. STEPHENS—Chairman
(165 North 19th Street, East Orange, N. J.)
R. R. ZIPPRODT—Secretary
(Associate Professor Civil Engineering, Columbia University, New York, N. Y.)

PAUL ANDERSEN	A. BURTON COHEN	T. D. MYLREA
R. L. BERTIN	J. DI STASIO	N. M. NEWMARK
H. P. BIGLER	ABRAHAM EPSTEIN	JOHN R. NICHOLS
A. J. BOASE	W. H. GRAVELL	L. H. NISHKIAN
R. D. BRADBURY	VERNON P. JENSEN	J. M. OXLEY
MILES N. CLAIR	N. D. MITCHELL	DEAN PEABODY, JR.

F. E. RICHART — HALE SUTHERLAND — C. A. WILLSON
W. C. SPIKER — W. S. THOMSON — R. B. YOUNG
W. H. WHEELER

This committee has its sub-committees still at work on proposed revisions in the tentative building regulations of 1936.

507—Recommended Practice for the Design of Metal Supports for Reinforcements—WILLIAM F. ZABRISKIE—Chairman
(13700 Sherwood Ave., Detroit)

CARL S. BARRY — D. H. BITNEY — MARTIN STELJES
W. E. WHITE

The assignment of this committee is implicit in the title.

Department 600, Field Construction

R. B. YOUNG, Chairman

604—Winter Concreting Methods—R. B. YOUNG—Chairman
(8 Strachan Ave., Toronto, Ont.)

JOHN E. DEIGNAN — WILFRID SCHNARR — C. E. WUERPEL
R. C. JOHNSON — A. G. TIMMS

This committee is assigned to write recommended practice for winter concreting.

609—Vibration of Concrete—ARTHUR RUETTGERS—Chairman
(Bureau of Reclamation, Denver)

ALFRED E. LINDAU—Vice-Chairman

H. L. FLODIN—Secretary
(Portland Cement Assn., 33 West Grand Ave., Chicago)

MILES N. CLAIR — H. S. MEISSNER — F. V. REAGEL
H. E. DAVIS — BEN MOREELL — L. C. ROGERS
R. E. DAVIS — D. E. PARSONS — LEWIS H. TUTHILL
C. M. HATHAWAY — O. G. PATCH — MORTON S. WESTLUND
F. H. JACKSON — C. W. PIERCE — M. O. WITHEY

As reorganized this committee has some fresh responsibilities. It had already brought in a report under former Chairman A. E. Lindau with such recommendations as the committee could venture from available data on the use of vibration as a means of placing concrete. Experience indicates that additional research data are needed and it is expected that the new committee may be in position to supply the data, for the express purpose of making recommendations more accurate and more explicit as to the use of vibration.

611—Inspection of Concrete—J. W. KELLY—Author-Chairman
(Engineering Materials Laboratory, University of Calif., Berkeley)

R. L. BERTIN — F. H. JACKSON — LEWIS H. TUTHILL
MILES N. CLAIR — ARTHUR R. LORD — R. C. WILSON
H. M. HADLEY — T. C. POWERS — R. B. YOUNG
ARTHUR RUETTGERS

Mr. Kelly has written for the Institute a Manual of Concrete Inspection. First and second drafts were reviewed and criticized in

detail by members of the committee. The third draft is now in process of similar review. It is a considerable document—probably will be in demand in and out of the Institute when it finally becomes available. It not only tells the concrete inspector what he should do but it sets down the underlying reasons why it should be done that way. It will be pretty much a fundamental education in the means to good concrete for many others than the inspector for whom it is primarily intended. But being a very considerable document it has moved slowly through the process of criticism and revision. It appears now to be near completion.

612—Recommended Practice for Curing Concrete—MARK MORRIS—Chairman
(Iowa State Highway Commission, Ames, Ia.)

R. W. CARLSON	F. C. LANG	STANTON WALKER
F. H. JACKSON	F. V. REAGEL	

This committee is assigned to write a recommended practice for curing concrete. There are many scattered data on the subject of curing but little in the way of specific instructions between two covers as a guide to such practice in curing as will make good on the design of the mix and good practice in placing.

613—Recommended Practice for the Design of Concrete Mixes—R. F. BLANKS—Chairman
(Bureau of Reclamation, Denver, Colo.)

HERBERT COFFMAN	W. F. KELLERMANN	H. C. ROSS
W. M. DUNAGAN	GUY H. LARSON	I. L. TYLER
FRED HUBBARD	T. C. POWERS	E. N. VIDAL
F. H. JACKSON	E. B. RAYBURN, JR.	

This committee has a difficult task—to present a recommended practice, probably with alternative methods, for the design of concrete mixtures, for strength, workability, durability and economy. The importance of these general factors and their relation to the general method of design to be defined.

614—Recommended Practice in Measuring, Mixing and Placing Concrete—LEWIS H. TUTHILL—Chairman
(Metropolitan Water District of Southern Calif., Los Angeles, Calif.)

JOHN G. AHLERS	LION GARDINER	DONALD D. REYNOLDS
C. F. BALL	BYRON A. HILL	R. W. SPENCER
E. W. BAUMAN	C. S. JOHNSON	H. F. THOMSON
SAM COMESS	A. LEVISON	I. L. TYLER
R. V. CREPPS	O. G. PATCH	R. B. YOUNG

The job of this committee is implicit in the title. Mr. Tuthill prepared a report and it had a very thorough going-over by practically all the members of the committee. He has extensively revised the report and it is now in the hands of the Institute Standards Committee.

616—Recommended Practice for the Application of Paint to Concrete Surfaces—R. E. COPELAND—Chairman
(Portland Cement Assn., 33 West Grand Ave., Chicago)

T. H. CHISHOLM	C. H. JUMPER	E. W. SCRIPTURE, JR.
H. A. GARDNER	JOHN MARSHALL	CLARA SENTEL
R. H. HALLET	T. J. PETERSEN	E. VIENS
	WAYLAND W. RICE	

What this committee has been asked to do is told in the title, to write a recommended practice for the application of paint of different kinds to concrete surfaces.

617—Specifications and Recommended Practice for Concrete Pavement and Base—H. F. CLEMMER, Chairman

R. D. BRADBURY	FRED HUBBARD	STANTON WALKER
A. T. GOLDBECK	F. H. JACKSON	W. H. WOOD
	BAILEY TREMPER	

What this committee has been asked to do is obvious from the title, to write specifications and recommended practice for concrete pavement and base.

618—Specifications and Recommended Practice for the Construction of Terrazzo Floors—VERNE O. MCCLURG—Chairman

JOSEPH CHUBB	A. J. RENNEN	C. G. WALKER
LOUIS DEL TURCO	CLIFFORD S. STEPHENS	H. H. WAPLES

Old as is the art of terrazzo there are no generally recognized standards. This committee is representative of those who specify, those who furnish materials, those who do the work and has been asked to write specifications and recommended practice for the construction of terrazzo floors.

Department 700, Shop Manufacture

W. G. KAISER, Chairman

711—Precast Floor Systems for Houses—F. N. MENEFEE—Chairman
(104 West Engineering Bldg., University of Michigan, Ann Arbor)

R. E. COPELAND	H. B. HEMB	J. W. WARREN
C. G. DUNNELLS	GLENN MURPHY	R. R. ZIPPRODT
	JOHN STRANDBERG	

There has been an increasing interest and activity in the field of manufacture and use of precast units for light load floor systems, involving joists and to some extent slabs. This committee has been asked to bring in a report which would cover the major essentials of the design of such floor systems and the means of getting the results in manufacture and construction. The work on a preliminary report is nearing completion.

714—Recommended Practice for the Construction of Concrete Silos—C. A. HUGHES—Chairman
(University of Minnesota, Minneapolis)

J. W. BARTLETT	WILLIAM GURNEY	D. G. MILLER
WALTER BRASSERT	W. G. KAISER	C. C. MITCHELL
CLAUDE DOUTHETT	G. L. LINDSAY	CHARLES F. ROGERS
H. B. EMERSON	J. W. MCCALMONT	STANLEY WITZEL

This committee came into being more or less spontaneously as the result of a definite need among the manufacturers of concrete silo staves and the builders of concrete silos to meet the agricultural demand for enduring structures against the various destructive agencies which lurk in the use to which silos are put, varying with the kind of ensilage. The committee is earnestly at work to produce a recommended practice for the construction of concrete silos of all kinds and will probably recommend higher standards.

Department 800, Special Requirements

R. E. DAVIS, Chairman

804—Wearing Surfaces for Heavy Duty Floors—J. FRUCHTBAUM—Chairman
(613 Root Bldg., Buffalo, N. Y.)

JOHN G. AHLERS	WALTER HUGHES	T. C. KNIGHT
STACEY CAMPBELL	MORGAN B. KLOCK	HARRY LORING

This committee's first report has been published. The preparation of further report which would be proposed for adoption as an A. C. I. standard, is the next step. For clearer emphasis on the committee's specific task its name has been changed as now shown.

Department 900, Joint Efforts

F. H. JACKSON, Chairman

Committee members here listed are the Institute's representatives on joint committees or committees of organizations other than A. C. I.

903—Joint Committee on Standard Specifications for Concrete and Reinforced Concrete—S. C. HOLLISTER—Chairman
(Cornell University, Ithaca, N. Y.)

JOHN G. AHLERS	F. H. JACKSON	BEN MOREELL
	N. M. STINEMAN	

904—Sectional Committee on Plastering—J. C. PEARSON

905—Sectional Committee on Specifications for Portland Cement—JOHN G. AHLERS, W. K. HATT, M. A. SWAYZE

906—Sectional Committee on Methods of Tests for Road Materials—R. W. CRUM

907—Highway Research Board—P. H. BATES, H. J. GILKEY, alternate

908—Committee on Masonry and Reinforced Concrete of the Structural Division, A. S. C. E.—RAYMOND E. DAVIS

910—Committee on Corelation of Research in Mineral Aggregates of the Highway Research Board—R. L. BERTIN, S. C. HOLLISTER

912—Sectional Committee on Sieves for Testing Purposes—J. C. PEARSON

913—Definitions and Nomenclature—(Sub-Com. V. of A. S. T. M. Com. C-9, becomes (1939) by special arrangement a joint committee of A. S. T. M. and A. C. I. reporting to both organizations. CLOYD M. CHAPMAN, Chairman, A. T. GOLDBECK, L. W. TELLER, H. J. LOVE, A. N. TALBOT, STANTON WALKER.

914—Sectional Committee, American Standards Association on Excavations and Foundations of the Building Code Corelating Committee—R. R. ZIPPRODT, R. L. BERTIN, alternate

915—A. S. C. E. Committee on Bearing Value of Pile Foundations—W. F. WAY

916—Standing Committee on Sizes of Coarse Aggregate—Simplified Practice Div., National Bureau of Standards—F. V. REAGEL.

Awards

The Wason Medal "for the Most Meritorious Paper"

Founded 1917 by Leonard C. Wason (deceased) who was president, American Concrete Institute 1915-1916. Awards are made by the Board of Direction. From 1917 to 1930 these were "for the most meritorious paper" presented at the previous annual convention. Beginning 1931 "for the most meritorious paper" in the *Proceedings* Volume of the previous year.

1916 Paper—A. B. McDaniel, "Influence of Temperature on the Strength of Concrete."

1917 Paper—Charles R. Gow, "History and Present Status of the Concrete Pile Industry."

1918 Paper—Duff A. Abrams, "Effect of Time of Mixing on the Strength and Wear of Concrete."

1919 Paper—W. A. Slater, "Structural Laboratory Investigations in Reinforced Concrete Made by Concrete Ship Section, Emergency Fleet Corporation."

1920 Paper—W. A. Hull, "Fire Tests of Concrete Columns."

1921 Paper—H. M. Westergaard, "Moments and Stresses in Slabs."

1922 Paper—George E. Beggs, "An Accurate Mechanical Solution of Statically Indeterminate Structures by Use of Paper Models and Special Gauges."

1923 Paper—John J. Earley, "Building the Fountain of Time."

1924 Two Papers—Richard L. Humphrey, "Twenty Years of Concrete" and "The Promise of Future Development."

1925 Paper—E. A. Dockstader, "Report of Tests Made to Determine Temperatures in Reinforced-Concrete Chimney Shells."

1926 Paper—A. Burton Cohen, "Correlated Considerations in the Design and Construction of Concrete Bridges."

1927 Paper—Arthur R. Lord, "Notes on Concrete—Wacker Drive, Chicago."

1928 Paper—Franklin R. McMillan, "Concrete Primer."

1929 Paper—L. G. Lenhardt, "The Concrete Lining of Detroit Water Tunnels."

1930 Paper—I. E. Burks, "Concreting Methods at the Chute a Caron Dam."

1931 Paper—Raymond E. Davis and Harmer E. Davis, "Flow of Concrete under the Action of Sustained Loads."

1932 Paper—Charles S. Whitney, "Plain and Reinforced Concrete Arches."

1933 Paper—L. Boyd Mercer, "Sliding Form Work."

1934 Paper—Raymond E. Davis, R. W. Carlson, G. E. Troxell and J. W. Kelly, "Cement Investigations for Boulder Dam with the Results up to the Age of One Year."

1935 Paper—Hardy Cross, "Why Continuous Frames?"

1936 Paper—George A. Maney, "Analysis of Multiple Span Rigid Frame Bridges by the Slope Deflection Method."

1937 Paper—R. B. Young, "Concrete—It's Maintenance and Repair."

1938 Paper—Wilbur M. Wilson and Ralph W. Kluge, "Tests of Rigid Frame Bridges."

The Wason Medal for Noteworthy Research Reported to the Institute

S. C. Hollister, for advancing the art of bridge construction in the design, construction and test of a concrete skew arch as reported in his 1928 paper: "The Design and Construction of a Skew Arch."

H. F. Gonnerman and P. M. Woodworth for the work reported in their 1929 paper: "Tests of Retempered Concrete."

M. O. Withey, for the work reported in his 1931 Institute paper, "Long Time Tests of Concrete."

T. C. Powers, for work reported in his 1932 Institute paper, "Studies of Workability of Concrete."

A. G. Timms and N. H. Withey, for the work reported in their 1934 Institute paper, "Temperature Effects on Compressive Strength of Concrete."

Ben Moreell, Douglas E. Parsons, A. H. Stang for the work reported in two 1935 Institute papers involving studies of Mesnager Hinges ("Articulations for Concrete Structures—the Mesnager Hinge" by Moreell and "Tests of Mesnager Hinges" by Parsons and Stang.)

J. R. Shank, for work reported in his 1936 Institute paper, "The Mechanics of Plastic Flow of Concrete."

F. E. RICHART and T. A. OLSON for the work reported in their 1937 paper "Rapid and Long-Time Tests of Reinforced Concrete Knee Frames."

THOMAS E. STANTON and LESTER C. MEDER, for the work reported in their 1938 paper "Resistance of Cement to Attack by Seawater and by Alkali Soils."

The Turner Medal

Founded 1927 by Henry C. Turner, New York, past-president, American Concrete Institute, a gold medal will be awarded not oftener than once a year "for notable achievement in or service to the concrete industry."

Awarded, 1928, to ARTHUR N. TALBOT, for "outstanding contributions to the knowledge of reinforced concrete design and construction."

Awarded, 1929, to WILLIAM K. HATT, for "pioneer work in reinforced concrete research; for a quarter century of devoted, outstanding and continuous service in developing the knowledge of concrete."

Awarded, 1930, to F. E. TURNEAURE, for "distinguished service in formulating sound principles of reinforced concrete design."

Awarded 1932 to DUFF A. ABRAMS, for "the discovery and statement of important fundamental principles governing the properties of concrete and reinforced concrete."

Awarded 1934 to JOHN J. EARLEY, for "outstanding achievement in developing concrete as an architectural medium."

Awarded 1939 to PHAON H. BATES, for "contributions to science, direction of research, and outstanding leadership in advancing the intelligent utilization of cement and concrete."

Institute Affairs

FISCAL YEAR JULY 1, 1938 TO JUNE 30, 1939
A REPORT BY THE SECRETARY-TREASURER FOR THE BOARD OF DIRECTION TO THE INSTITUTE MEMBERS

In the twelve months July 1, 1938 to June 30, 1939 the Institute gained 12 members and added $628.06 to its surplus. This status is best seen in relation to the figures for members, dollars and pages published for other years in Tables 1 and 2.

TABLE 1—TREND OF AMERICAN CONCRETE INSTITUTE, 1926-1939

Fiscal Year Ended June 30,	Members	Pages Proceedings or Journal	Receipts	Expenditures	Surplus*
1926	1935	796	26,115.44	27,291.48	4,250.47
1927	2315	724	29,373.51	26,862.73	6,761.25
1928	2530	882	37,867.35	32,552.01	12,076.59
Mar. 31, 1929	2736				
1929	2616	880	39,899.01	34,446.39	17,529.21
1930	2565	1278	39,837.84	36,902.31	20,464.74
1931	2438	1996	36,871.64	43,154.47	14,181.91
1932	1970	1112	28,839.51	32,490.58	†10,169.59
1933	1389	598	19,347.81	20,680.28	8,837.12
1934	1154	700	18,713.23	17,414.59	‡8,990.76
1935	1161	688	18,553.17	19,653.67	7,890.26
1936	1214	904	20,457.08	20,383.70	7,963.54
1937	1416	**936	22,585.10	24,863.20	5,685.54
1938	1559	***1090	24,555.20	24,552.84	5,687.90
1939	1571	1072	25,204.66	24,576.60	6,315.96

*Surplus includes cash on hand and book value (cost) of securities less accounts payable but does not include accounts receivable.

**Includes supplement issued Sept.-Oct. 1937.

***Includes Supplement September 1938. About half the page increase as compared with 1937 is due to including Directory as a part of Annual Volume.

†Reduction of cash	$3,651.07	‡Margin of Receipts over Exp.	$1,298.64
Loss on securities sold	361.25	Loss on securites sold	1,145.00
Total Decrease in surplus	$4,012.32	Net Increase in surplus	$ 153.64

TABLE 2—A. C. I. MEMBERSHIP, 1921-1939

End of Fiscal Year June 30,	Active	Contributing	Honorary	Student	Total
1921	598	90	..	..	688
1922	806	84	..	..	890
1923	887	77	..	..	964
1924	1191	98	..	..	1289
1925	1426	110	..	..	1536
1926	1819	116	..	..	1935
1927	2193	120	2	..	2315
1928	2407	121	2	..	2530
1929	2502	113	1	..	2616
1930	2455	109	1	..	2565
1931	2343	94	1	..	2438
1932	1898	62	10	..	1970

End of Fiscal Year June 30,	Active Members (Ind. Corp.)	Contributing	Honorary	Student	Total
1933	1319 (916 403)	48	10	11	1389
1934	1099 (931 168)	41	9	5	1154
1935	1102 (970 132)	37	10	12	1161
1936	1147 (1016 131)	51	9	7	1214
1937	1328 (1179 149)	69	8	11	1416
1938	1450 (1296 154)	76	8	25	1559
1939	1468 (1330 138)	71	8	24	1571

Most important is the trend of publications in subject matter and treatment toward a greater recognition of the interest in and importance of papers, committee reports and the contents of the new (since Nov. 1938) Job Problems and Practice Department, which meet a demand for "information for use"—contributions which translate into terms of design, construction and manufacturing practice the increased knowledge at our disposal. Several Institute committees organized to give that kind of service to members have recently made such progress that their reports seem near the stage of release for publication.

Honorary Members

Elected by the Board of Direction

Adolph Buhler
Fritz Emperger
William K. Hatt
Alfred E. Lindau
Arthur N. Talbot
Sanford E. Thompson
F. E. Turneaure
Henry C. Turner

Richard L. Humphrey (Deceased)
Edward D. Boyer (Deceased)
R. W. Lesley (Deceased)
Leonard C. Wason (Deceased)

The list of members of the American Concrete Institute as of July 1, 1939, includes after each name, as usual, the year from which the membership dates. There is also, as prescribed by the Board of Direction, a recognition of contributions to Institute work, in a record of (1) Institute Papers published; (2) Chairmanships and Secretaryships of Technical Committees whose reports were published; (3) Service as member of an Administrative committee; (4) Service as a member of the Board of Direction (5) Winner of an Institute Medal; (6) Election to Honorary Membership. These records are set down after careful search of Institute files. The Secretary-Treasurer will be grateful for definite note of errors of omission or commission which have crept into this record.

The Membership July 1, 1939 is as follows:

Honorary Members	8
Members—	
Individuals	1330
Corporations	138
Contributing	71
Student	24
Total	1571

MEMBERS

AMERICAN CONCRETE INSTITUTE

Diligent effort is made to keep the official membership card list corrected to date in every detail. This printed list is compiled from that with corrections to July 1, 1939. Contributing members' and corporation members' names are in capitals and the former are further designated by a (★). Both these membership classes are cross-indexed with listings of their representatives. If errors have crept in please notify us promptly. Names and addresses of new members will be published in the JOURNAL.

—SECRETARY-TREASURER

Abberley, Elbert K., (*1927*) Demarest, N. J.

Abe, Mikishi, (*1920*) No. 10, Miyamura-Cho, Azabu-Ku, Tokyo, Japan

Abeles, Paul, (*1938*) III Seidlgasse 39, Vienna, Austria-Germany.

Abrams, Duff A., (*1910*) 11 West 42nd St., New York, N. Y.

Co-author: (with A. N. Talbot) "Method of Testing Cement Pipe" V. 8, p. 713; Author: "Effect of Time of Mixing on the Strength and Wear of Concrete" V. 14, p. 22 (Wason Medal for most meritorious paper, 1918); "Effect of Vibration, Jigging and Pressure on Fresh Concrete" V. 15, p. 63; "Flexural Strength of Plain Concrete" V. 18, p. 20; "Proportioning Concrete Mixtures" V. 18, p. 174; "Making Good Concrete" V. 20, p. 175; "Tests of Impure Waters for Mixing Concrete" V. 20, p. 422; "Water-Cement Ratio as a Basis of Concrete Quality," V. 23, p. 452; "The American Concrete Institute and Concrete Research" V. 27, p. 953. Sec'y, Com. on Research, V. 17, p. 281; Sec'y. Com. E-3—Research, V. 18, p. 198; Member: Publications Committee, 1929; Board of Direction: Director, 1921-26; V. Pres., 1927-29; Pres., 1930-31; Past-Pres., 1932-38. Awarded (1932) Turner Medal for "The discovery and statement of important fundamental principles governing the properties of concrete and reinforced concrete."

Adams, Wm. (See Stuart's Granolithic Co., Ltd.)

Adler, Julius (*1935*) 2001 Architects Bldg., 17th & Sansom Sts., Philadelphia, Pa.

Affleck, B. F., (*1908*) 210 S. La Salle St., Chicago, Ill.

Member Board of Direction: Director, 1914; Director-at-Large 1934---

Agramonte, Albert A., (*1923*) Simbron 3191, Buenos Aires, Argentina

Ahlers, John G., (*1920*) Ahlers Construction Co., 205 W. 34th St., New York, N.Y.

Author: (with Stanton Walker) "Field Tests of Concrete" V. 20, p. 358; (with J. J. Lindon and Millard F. Bird) "Wear Tests on Finishes of Industrial Concrete Floors" V. 25, p. 778; "Report on Field Tests and Methods Used in Building Construction to Obtain Concrete of Specified Strength" V. 19, p. 114; "New Experiences in Concrete Control" V. 22, p. 159; "Fire Damage to and Repair of a Concrete Factory Building" V. 26, p. 748; "Selecting Concrete Plant, Meadowbrook Hospital" V. 30, p. 142; "Experiences of an American Contractor in London, England" V. 33, p. 613. Ch. Com., C-1, Contractor's Plant, V. 19, p. 360; V. 20, p. 648; Ch. Com. 802, "Good Practice in Concrete Floor Finish" V. 26, p. 520; Member: Advisory Com., 1926-28; Board of Direction, Director, 1926-29 and 1932-34; Director-at-Large, 1934-35.

Ahrens, Henry, (*1936*) 4707 39th St., Queens Borough, New York, N. Y.

Airey, Edwin, (*1925*) Governing Director, William Airey & Sons, Ltd., Eldon House, Leeds 2, England

Aisher, O. A., (*1932*) The Marley Tile Co., Ltd., Riverhead, Sevenoaks, Kent, England.

Alba, Fernando de (*1938*) Colima 294, Mexico, D. F., Mexico.

Alessandri A., Carlos (*1938*) Huerfanos 1294 of 51, Santiago, Chile

Alexander, E. C., (*1927*) Massey Concrete Products Co., 939 Peoples Gas Bldg., Chicago, Ill.

Alexander, J. B. (See Southwestern Portland Cement Co.)

Alin, A. L., (*1920*) U. S. Engineer Office, Denison, Texas.

Allan, W. D. M., (*1928*) Portland Cement Assn. 33 W Grand Ave., Chicago, Ill.
Author: (with R. E. Copeland) "Technological Developments in Fireproof Concrete Homes" V. 29, p. 351. "The Possibilities and Limitations of Present and Future Markets for Concrete Building Units" V. 18, p. 155; "Standards of Performance of Concrete For Stucco" V. 25, p. 794; "Shrinkage Measurements of Concrete Masonry" V. 26, p. 699; "Shrinkage Measurements of Concrete Block Masonry" V. 28, p. 177. Sec'y, Com. P-6, Concrete Products Plant Operation, V. 18, p. 260; Ch. Com. 401, "Portland Cement Stucco Finishes" V. 26, p. 29; Ch. Com. 408, "The Use of Color in Concrete" V. 27, p. 975; Ch. Com., 709, "Specification for Concrete Burial Vaults" (Proposed) V. 27, p. 1251 (Tentative) V. 28, p. 633. Member Advisory Com. 1930-33.

Allbright, Edwin F., (*1928*) Grid Flat Slab Corp., 761 Dudley St., Boston, Mass.

Allen, Cleveland (*1924*) 11 Woods Edge Road, West Medford, Mass.

Allen, H. S.(*1927*) 13th & Holmes St., North Kansas City, Mo.

Allen, John E., (*1925*) 71 Church St., Winchester, Mass.

Allen, O. T., (*1920*) American Steel & Wire Co., 208 S. LaSalle St., Rm. 1422, Chicago, Ill.

ALLENTOWN PORTLAND CEMENT CO., (*1910*) Catasauqua, Pa. (R. S. Weaver)

***ALLENTOWN PORTLAND CEMENT CO.,** (*1936*) 128 Bridge St., Catasauqua, Pa. (F. A. Weibel, Secy. & Treas.)

***ALPHA PORTLAND CEMENT CO.,** (*1905*) Easton, Pa. (Louis Anderson, Jr.)

Altman, Frank S., (*1935*) 814 Northwestern Bank Bldg., Minneapolis, Minn.

Alves de Lima, Jorge, (*1929*) 51 Rua Libero Badaro, Sao Paulo, Brazil

Alvord, Henry B., (*1925*) Northeastern University, 360 Huntington Ave., Boston, Mass.
Administrative Assistant, A. C. I. 1917; Sec'y, 1918-19.

Amann, Dorothy A., Librarian, Southern Methodist University, Dallas, Texas.

AMBURSEN CONSTRUCTION CO., INC., (*1927*) 295 Madison Ave., New York, N. Y. (L. A. Robb)

***AMERICAN AGGREGATE CO.,** (*1936*) 715 National Fidelity Bldg., Kansas City, Mo. (J B. Cleary, Gen. Mgr.)

AMERICAN BUILDERS SUPPLY CO., (*1930*) 405 Garden St., Louisville, Ky. (Jas. F. McCracken)

AMERICAN CAN CO., (*1914*) 230 Park Ave., New York, N. Y. (C. G. Preis)

AMERICAN CONCRETE PIPE ASSOCIATION, (*1922*) 33 West Grand Ave., Chicago, Ill. (M. W. Loving)

AMERICAN CONCRETE & STEEL PIPE CO., (*1937*) P. O. Box 1428, Arcade Station, Los Angeles, Calif. (H. H. Jenkins, Vice-Pres.)

AMERICAN ISTEG STEEL CO., (*1939*) 120 Broadway, New York, N. Y. (J. Mercadante)

Amirian, Thomas F., (*1937*) 31 Grove St., Boston, Mass.

Anaya y S, Manuel (*1938*) Comision Nacional de Irrigacion, Angostura, Son., Mexico

Anderson, F. Paul, (*1939*) Research Engineer, Dept. of Highways, Testing Lab., Frankfort, Ky.

Anderson, Jr., Louis (See Alpha Portland Cement Co.)

Andersson, David, (*1924*) Ch. Engr. Aktiebolaget Armerad Betong, Malmo, Sweden

Andrews, L. E., (*1928*) 634 W. State St., Trenton, N. J.

Anonas, Gregorio (See Cebu Portland Cement Co.)

ANTI-HYDRO WATERPROOFING CO., (*1921*) 265 Badger Ave., Newark, N. J. (M. W. Meyer)

Arango, Henrique G., (*1931*) P. O. Box 846, Panama City, Panama

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Asarpota, D. M., (*1936*) Ellis Bridge, Ahmedabad, India.

***ASH GROVE LIME & PORTLAND CEMENT CO.,** (*1936*) Chanute, Kansas. (A. D. Conrow)

ASSOCIATED CEMENT COMPANIES, LTD., THE, (*1937*) Esplanade House, Waudby Road, Bombay, India.

ASSOCIAZIONE ITALIANA PER GLI (*1927*) Studi sui Materiali da Costruzione, Istituto di Scienze delle Construzioni, Via Eudossiana, Rome, Italy

Ayers, Joseph W. (See C. K. Williams & Co.)

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Babcock, Charles D., (*1937*) Cattaraugus, N. Y.

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Bahmeier, H. F., (*1938*) c/o Bureau of Reclamation, Bayfield, Colo.

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Ball, Donald G., (*1938*) 36 Middlesex St., Winchester, Mass.

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Bannister, Kenneth (See Cemento de Mixcoac, S. A.)

Barclay, J. B., (*1928*) Northern Construction Co., Ltd., Box 879, Prince Rupert, B. C., Canada

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Barney, W. E., (*1934*) South Park, Ohio

***BARNEY CORP., W. J.,** 101 Park Ave., New York, N. Y. (Wm. Joshua Barney)

Barney, Wm. Joshua (See W. J. Barney Corp.)

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Bartholomew, Tracy (See Duquesne Slag Products Co.)

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Bashore, H. W., (*1937*) 3930 Huntington, N. W., Washington, D. C.

Bateman, John Henry, (*1938*) College of Engineering, Louisiana State University, University, La.
Co-author (with Alfred H. White) "Soaps as Integral Waterproofings for Concrete" V. 22, p. 535.

Bates, P. H., (*1926*) National Bureau of Standards, Washington, D. C.
Author: (with J. R. Dwyer) "Cement as a Factor in the Workability of Concrete" V. 24, p. 43; (with C. H. Jumper) "Notes on the Progress of Some Studies on Crazing of Portland Cement Mortars" V. 24, p. 179; (with R. L. Blaine) "Notes on Hardening Cements at the Boiling Point of Water" V 28, p. 531; "Review of the Present Status of Iron Portland Cement" V. 8, p. 566; "The Constitution of Portland Cement. Some

Results Obtained at the Experimental Cement Plant of the Bureau of Standards" V. 9, p. 368; "The Properties of Portland Cement Having a High Magnesia Content," V. 10, p. 470; "Some Further Results Obtained in Invesitgating the Properties of Portland Cements Having a High MgO Content" V. 10, p. 89; "Crazing on Cement Products" V. 21, p. 126; "Why Time is a Factor in the Study and Use of Cement" V. 23, p. 436; "Trends in the Production and Use of Various Types of Hydraulic Cements" V. 31, p. 225; "At the Forks" V. 32, p. 401. Ch. Com., 202. "Variations in Standard Portland Cement" V. 26, p. 65. Member: Publications Com., 1929-34, Ch. 1937-; Program Com., 1934 and 1937-, Ch. 1930-32; Advisory Com., 1930-34 and 1937-; Standards Com., 1937-; Board of Direction, Director, 1926-31; V. Pres., 1932-33; Pres., 1934-35 (July 1934-Feb. 1936); Past Pres., 1936-. Awarded (1939) Turner Medal for "Contributions to science, direction of research, and outstanding leadership in advancing the intelligent utilization of cement and concrete."

Battey, Paul L., (*1937*) Battey & Childs, Inc., 231 So. LaSalle St., Chicago, Ill.

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Baumberger, C. (See San Antonio Portland Cement Co.)

Baylor, H. D. (See Louisville Cement Co.)

Bayot, Jean M., (*1935*) Apartado 663, Caracas, Venezuela

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Co-author: (with H. J. Gilkey and S. J. Chamberlin) The Bond Between Concrete and Steel," V. 35, p. 1.

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Author: "Rigid Frame Concrete Bridges" V. 30, p. 196.

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Besozzi, Leo, (*1934*) 314 Hammond Bldg., Hammond, Ind.

BESSER MANUFACTURING COMPANY, (*1937*) Alpena, Mich. (J. H. Besser, Pres.)

Besser, J. H. (See Besser Mfg. Co.)

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Author: "Notes on Inspection of Structures in Europe" V. 33, p. 521; "Cooperation Needed for Architectural Concrete" V. 35, p. 365. Member: Advisory Com., 1935-.

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Boulton, H. S., (*1938*) 2323 Asbury Ave., Evanston, Ill.

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Brassert, Walter, (*1919*) Bloomfield, Ind.
Ch. Com., P-4, Concrete Staves, V. 18, p. 253; V. 20, p. 657; V. 22, p. 662.
662.

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BROKEN HILL PROPRIETARY CO., LTD., (*1925*) Box 196, Newcastle, N. S. W., Australia (L. Bradford, Mgr.)
Brookman, Roger S., (*1937*) 135 Roosevelt Road, Rochester, N. Y.
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Brown, Elwood H., (*1933*) 2416 McKinley Ave., Berkeley, Calif.
Brown, George D., (*1936*) Florida Portland Cement Co., Tampa, Fla.
Brown, Jr., Harlan E., (*1937*) 524 Pittock Block, Portland, Ore.
Brown, Maurice F., (*1937*) 44 School St., Boston, Mass.
Brown, Phillip P., (*1918*) 35 Chatsworth Road, Croyden, England
Brown, Ralph Charles, (*1937*) 1103 Highland Drive, Chattanooga, Tenn.
Brown, W. Durand, (*1939*) 410 Murphy Bldg., Detroit, Mich.
Browne, Nolan, (*1935*) Nolan Browne Co., 1000 Eagle Ford Road, Dallas, Texas
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Ch. Com., P-7, Concrete Pipe, Drain Tile and Conduit, V. 18, p. 277; V. 20, p. 669; V. 21, p. 583.
BUFFALO SLAG CO., (*1922*) 1070 Ellicott Square, Buffalo, N. Y. (Carlton S. Wicker, Mgr.)
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Author: "Construction of the Grandfey Viaduct" V. 23, p. 489.
Bullen, C. H., (*1938*) 228 No. LaSalle St., Chicago, Ill.
Author: "Construction of the San Jacinto Memorial" V. 34, p. 421.
Bunce, E. H., (*1933*) New Jersey Zinc Co., Palmerton, Pa.
Bunger, H. P., (*1937*) U. S. Bureau of Reclamation, Balboa Heights, Canal Zone, Panama.
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Butler, Gordon H. (See Polaris Concrete Products Co.)
Butler, Morgan R., (*1922*) Butler Bin Co., 154 Wisconsin Ave., Waukesha, Wis.
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CALAVERAS CEMENT CO., (*1936*) 315 Montgomery St., San Francisco, Calif. (William Wallace Mein, Jr.)

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CANADIAN FOREIGN INVESTMENT CORP., LTD., (*1934*) Room 805, 1111 Beaver Hall Hill, Montreal, Que., Canada

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CEBU PORTLAND CEMENT CO., (*1937*) Cor. Azcarraga & Evangelista Sts., Manila, P. I. (Gregorio Anonas)

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***CHAIN BELT CO.,** (*1936*) Milwaukee, Wis. (Chas. F. Ball)

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Author: "Results of Experiments upon Effect of Sea Water on the Tensile Strength of Various Mixtures of Cement and Sand" V. 6, p. 172; "Waterproofing Concrete Without Altering Its Appearance" V. 6, p. 587; "The Effect of Electrolysis on Metal Embedded in Concrete" V. 7, p. 647; "Waterproofing with Water" V. 7, p. 672; "Tests of Waterproofing for Concrete" V. 9, p. 250; "Data on Lime Putty and Cream of Lime" V. 10, p. 459 and 465; "The Use of the Universal Sand Tester" V. 12, p. 481. Ch. Com., "Treatment of Concrete Surfaces" V. 12, p. 473; Sec'y, Com., E-5, Concrete Aggregates, V. 16, p. 230. Member: Board of Direction, Director, 1930-31.

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Author: "Cost and Value of Concrete Pavements" V. 5, p. 356.

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Ch. Com., 607, report 1932 "The Use of Cement in Bulk" V. 28, p. 37.

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Fleming, A. G., (*1932*) Canada Cement Co., Ltd., Montreal, Que., Canada

Fleming, E. M., (*1937*) Portland Cement Assn., 347 Madison Ave., New York, N. Y.

Fleming, T. H., (*1938*) Concrete Transport Mixer Co., 650 Rosedale Ave., St. Louis, Mo.

Flesheim, S. W., (*1922*) The Master Builders Co., 7016 Euclid Ave., Cleveland, Ohio

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Flores, Manuel Gonzales, (*1936*) Calle de Ezequiel Montes III, Mexico, D. F., Mexico

**FLORIDA PORTLAND CEMENT CO.,* (*1936*) Box 1528, Tampa, Fla. (F. M. Traynor)

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Foster, L. J., (*1937*) Bureau of Reclamation, Yuma, Ariz.

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Author: "Tests of Blast Furnace Slag as the Coarse Aggregate in Concrete" V. 14, p. 95; "Purchasing Centrally Mixed Concrete" V. 25, p. 632. Sec'y Com., E-6, "Destructive Agents and Protective Treatments," V. 19, p. 301; V. 20, p. 564.

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Ch. Com. 804, "Concrete Wearing Surfaces for Floors" V. 35, p. 21.

Fuller, M. G., (*1938*) Bin 5, Vinita, Okla.

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Member: Board of Direction, Director, 1938.

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Author: "Factory-Made Reinforced-Concrete Products for Railway Purposes" V. 14, p. 250.

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Author: "Grading and Workability" V. 33, p. 319.

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Glover, V. L., (*1934*) Illinois Division of Highways, Bureau of Materials, 126 East Ash St., Springfield, Ill.

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Author: (with E. B. Smith) "Tests of Large Reinforced Concrete Slabs" V. 12, p. 324; "Tests of Circular and Egg-Shaped Reinforced Concrete Sewer Pipe" V. 11, p. 240; "The Influence of Total Width on the Effective Width of Reinforced-Concrete Slab Subjected to Central Concentrated Loading" V. 13, p. 78; "Friction Tests of Concrete on Various Sub-Bases" V. 13, p. 239; "What Are the Most Significant Tests for Concrete" V. 22, p. 386; "Gradation and Character of Aggregates as a Factor in Workability" V. 24, p. 56. Member: Advisory Com., 1922-23.

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Co-author: (with A. N. Talbot) "Test of an Eight year-old Flat-Slab Floor of the Western Newspaper Union Building" V. 14, p. 192; Author: (with F. E. Richart) "Test of a Flat-Slab Floor of the New Channon Building" V. 17, p. 182; (with P. M. Woodworth) "Tests of Retempered Concrete" V. 25, p. 344; (Wason Research Medal, 1929); (with A. G. Timms and T. G. Taylor) "Effect of Calcium and Sodium Chlorides on Concrete When Used for Ice Removal" (progress report) V. 33, p. 107.; "Some Tests on the Effect of Age and Condition of Storage on the Compressive Strength of Concrete" V. 14, p. 101; "Study of Methods of Curing Concrete" V. 26, p. 359. Ch. Com., E-3, Research, V. 22, p. 677; V. 23, p. 539; V. 24, p. 745; V. 25, p. 432; Ch. Com., 101, "Current Researches on Plain and Reinforced Concrete and Related Materials" V. 27, p. 469.

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Author: "Application of Concrete in Barge Canal Work" V. 6, p. 196.

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Author: "Reinforced Concrete Girder Bridges of Over 100-ft. Span" V. 35, p. 193.

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Co-author: (with Henry D. Dewell) "Flat Slabs and Supporting Columns and Walls Designed as Indeterminate Structural Frames" V. 34, p. 321.

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Author: "Reinforced Concrete" V. 3, p. 58; "Tests of Reinforced Concrete Hollow Tile Floor Spans" V. 4, p. 28; "Genesis of Reinforced Concrete Construction" V. 12, p. 21; "Instruction in Reinforced Concrete Construction" V. 13, p. 284; "Moment Coefficients for Flat-Slab Design with Results of a Test" V. 14, p. 164; "Tests of a Concrete Mixer" V. 17, p. 47; (with R. B. Crepps) "Tolerance of Coarse Aggregate Passing the ¼-in. Sieve as Affecting Specifications for Gravel Aggregate" V. 17, p. 213; "Note on the Ball Test Applied to Cement Mortar" V. 18, p. 52; "Note on Fatigue of Mortar" V. 18, p. 167; "Notes on Construction of Concrete Stadium" V. 20, p. 113; "Extensibility of Concrete" V. 22, p. 364. Ch. Com., E-3, Research, V. 17, p. 281; V. 18, p. 198. Member: Publications Com., 1915; Ch., 1920; Board of Direction: Director, 1914-15; V. Pres. 1915-16; Pres., 1917-19; Past Pres., 1920-29. Awarded (1929) Henry C. Turner Medal for "pioneer work in reinforced concrete research; for a quarter century of devoted, outstanding and continuous service in developing the knowledge of concrete."

Haubert, E. P. (See Lawrence Portland Cement Co.)

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Havelin, J. E. (See Philadelphia Electric Co.)

Havemeyer, J. F. (See Concrete Steel Co.)

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Hayden, Harold W., (*1937*) Librarian, Carnegie Library, Bucknell University, Lewisburg, Pa.

Hayden, J. F. (See Trinity Portland Cement Co., Dallas, Texas)

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Co-author: (with F. E. Richart) "Tests of Reinforced Concrete Columns Under Sustained Loading" V. 35, p. 33.

Henderson, G. H. (See Rhode Island Dept. of Public Works)

Henderson, H. B. (See Cowin & Co. Ltd.)

Henderson, Herbert, (*1935*) P. O. Box 1214, Pittsburgh, Pa.

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Co-author: (with E. A. Pisapia) "Factors of Workability of Portland Cement Concrete" V. 32, p. 641.

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Author: "Maintenance and Repair of Concrete Structures in Railroad Construction" V. 33, p. 251.

Hirsh, A. M. (See Lock Joint Pipe Co.)

Hirshfeld, C. F., (*1934*) Detroit Edison Co., Library, 2000 Second Ave., Detroit, Mich.

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Co-author: (with D. H. Pletta) "Beams with Intermediate Expansion Hinges in Rigid-Frame Bridges" V. 35, p. 149.

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Co-author: (with R. J. Wig) "Problems Arising in the Design and Construction of Reinforced-Concrete Ships" V. 14, p. 441. Author: "Plasticity and Temperature Deformations in Concrete Structures" V. 15, p. 127; "Construction of Concrete Barges for the New York State Barge Canal" V. 15, p. 289; "The Design and Construction of a Skew Arch" V. 24, p. 371; (Wason Research Medal); "Livability of Concrete Dwellings" V. 25, p. 801; "Specifications for The Small Job" V. 26, p. 477; "Studies of Concrete Mixtures" V. 27, p. 959; "Tests of Concrete from a Transit Mixer" V. 28, p. 405; "President's Address" V. 30, p. 247; "The Experimental Study of Stresses at a Crack in a Compression Member" V. 30, p. 361; "A Short Method for Computing Moments in Continuous Frames" V. 33, p. 147. Ch. Com., E-7, Waterproofing, V. 19, p. 294; V. 20, p. 641; Ch. Com., J-1, A. C. I. representation, Joint Com. on Concrete and Reinforced Concrete, V. 21, p. 328; Member: Advisory Com., 1922-25; Ch. 1926-29; Program Com., 1927 and 1930-31; Ch., 1928; Publications Com., 1929; Board of Direction, Director, 1923-27; V. Pres., 1928-31, Pres., 1932-33; Past Pres., 1933-.

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Author: (with A. S. Levens) "Strength and Shrinkage of Mortars Made with Blends of Portland Cement and Pozzulanic Materials" V. 27, p. 317.

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formity of Pavement Concrete" V. 33, p. 411; "Tests of Concrete Curing Materials" V. 35, p. 481. Sec'y, Com., E-5, Aggregates, V. 23, p. 573; V. 24, p. 777; V. 25, p. 656. Member: Advisory Com., 1929; Program Com., 1938-; Board of Direction, Director 1937-39.

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Sec'y, Com., E-8, Expansion Joints in Concrete Construction, V. 20, p. 619; V. 21, p. 321.
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Author: "Inundation as a Practical Aid to Uniform Concrete" V. 21, p. 216.

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Studies of Concrete and Aggregates" V. 30, p. 121; "Relation Between Quality and Economy of Concrete" V. 29, p. 325; "Lightweight Slag Concrete" V. 31, p. 1; "Effect of Brand and Type of Cement on Strength and Durability of Concrete" V. 31, p. 247; "A Study of Reinforcement in Concrete Slabs" V. 33, p. 1. Member: Publications Com., 1932 ; Board of Direction, Director, 1937-38.

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Author: "Analysis of Multiple Span Rigid Frame Bridges by the Slope Deflection Method" V. 32, p. 495; (Wason Medal for most meritorious paper); "Should the Type of Indeterminate Problem Determine Its Method of Solution" V. 34, p. 605.

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Ch. Com., E-4, Fire Resistance of Concrete, V. 21, p. 284; V. 25, p. 812.

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Author: "The Mortar Voids Method of Designing Concrete Mixtures" V. 29, p. 9.

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Ch. Com., S-3, Reinforced and Plain Concrete Sewers and Conduits, V. 19, p. 429; V. 20, p. 757; Ch. Com., E-8, Expansion Joints in Concrete Construction, V. 20, p. 619; V. 21, p. 321.

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Author: "Some Comparative Corrosion Tests of Plastered Metal Lath—A Progress Report" V. 10, p. 445; "Tests of Stucco" V. 14, p. 109; (with G. A. Smith) "Tests of Concrete Tanks for Oil Storage" V. 15, p. 186; "Specifications for U. S. Standard Sieve Series" V. 16, p. 49; (with J. J. Earley) "New Developments for Surface Treated Concrete and Stucco" V. 16, p. 70; "Shrinkage of Portland Cement Mortars and Its Importance in Stucco Construction" V. 17, p. 133; (with G. F. Loughlin) "An Interesting Case of Dangerous Aggregate" V. 19, p. 142; "Thoughts on Concrete Houses" V. 19, p. 167; (with F. A. Hitchcock) "Economic Value of Admixtures" V. 20, p. 312; "Properties and Problems of Masonry Cements" V. 28, p. 349; (with E. M. Brickett) 'Studies of High Pressure Steam Curing" V. 28, p. 537; (with H. G. Collins) "A Method for Determining the Air Content of Freshly Mixed Mortars and Concretes" V. 32, p. 298; "Future Policies of the Institute" V. 34, p. 377; (with R. F. Adams) "Low Head Permeability Tests of Mortar Pots" V. 35, p. 285. Ch. Com., Treatment of Concrete Surfaces, V. 14, p. 466; V. 15, p. 343; V. 16, p. 304; V. 17, p. 306; V. 18, p. 279; V. 19, p. 464; V. 21, p. 555. Member: Publications Com., 1930-31, 1937; Ch. 1932-36; Advisory Com., 1932-37; Program Com., 1932-37; Board of Direction, Director, 1921-23, 1928-34; V. Pres., 1934-36, Pres., 1937; Past Pres., 1938-.

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Author: "Reinforced Concrete Pavements and Roadways" V. 10, p. 112. Ch. Com., J-2, Concrete Pipe, report 1922, V. 18, p. 318;, Ch. Com., J-2, Joint Concrete Culvert Pipe, report 1923, V. 19, p. 292.

Peck, John Sanford, (*1936*) College of the City of N. Y., 138th St. & Convent Ave., New York, N. Y.

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Author: "Effect of the Florida Hurricane on Structures" V. 23, p. 276.

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PENNSYLVANIA-DIXIE CEMENT CORP., (*1937*) 60 E. 42nd St., New York, N. Y. (Joseph H. Chubb)

PENNSYLVANIA STATE HIGHWAY DEPT., (*1911*) Harrisburg, Pa. (S. W. Marshall)

Penny, A. F. (*1937*) c/o Sternson Structural Specialties Ltd., 126 Bruce St., Brantford, Ont., Canada

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Co-author (with W. H. Herschel) "Factors of Workability of Portland Cement Concrete" V. 32, p. 641.

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PITTSBURGH TESTING LABORATORY, (*1912*) Stevenson St. & Locust, Pittsburgh, Pa. (A. R. Ellis, Pres.)

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Ch. Com., S-1, Reinforced Concrete Chimneys, V. 18, p. 221.

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POLARIS CONCRETE PRODUCTS CO., (*1923*) Box 86, West Duluth, Minn. (Gordon H. Butler, Pres.)

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Author: "Reinforced Concrete as Applied to Monumental Buildings" V. 24, p. 105.

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Ch. Com., 406, "Economics of Light Weight Concrete in Buildings" V. 27, p. 925.

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Author: "Consistency Indicator for A Ready-Mixed Concrete Plant" V. 31, p. 105.

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RICHMOND SCREW ANCHOR CO., INC., (*1937*) 245 Bush St., Brooklyn, N. Y. (Chas. A. Snyder, Pres.)

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Co-author with L. J. Snyder "Thermal Properties of Mass Concrete" V. 30, p. 35.

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RIVERTON LIME AND STONE CO., (*1939*) Riverton, Va. (P. L. Rogers)

Robb, L. A. (See Ambursen Construction Co., Inc.)

Roberts, Dwight F., (*1937*) 920 Santee St., Los Angeles, Calif.

Roberts, G. L. (See United Carbon Co., Inc.)

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Robison, John W., (*1937*) U. S. R. B., Redding, Calif.

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SANTA CRUZ PORTLAND CEMENT CO., (*1922*) Davenport, Calif. (E. W. Rice)

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Author: "Bond, Shear and Diagonal Tension in Reinforced Concrete" V. 28, p. 187; "The Mechanics of Plastic Flow of Concrete" V. 32, p. 149; (Wason Research Medal). "Bond Creep and Shrinkage Effects in Reinforced Concrete" V. 35, p. 81.

Shanley, Edmund M., (*1938*) 2763 Madera Ave., Oakland, Calif.

Sharman, Basil C. (See John Ellis & Sons, Ltd.)

Shaw, Saul, (*1939*) 24 Walnut St., Newark, N. J.

Shedd, Thomas C., 306 Engineering Hall, Univ. of Ill., Urbana, Ill.

Sheets, Frank T., (*1938*) 33 W. Grand Ave., Chicago, Ill.
Author: "A Challenge—Shorten the Lag Between Research and Practice V. 34, p. 541.

Sheridan, Robert R., (*1935*) R. F. D. No. 1, Lake Road, Webster, N. Y.

Sherlock, R. H., (*1927*) 2025 Vinewood Ave., Ann Arbor, Mich.

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Sherman, John R., Yakima Cement Products Co., Yakima, Wash.

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Shields, H. C., (*1926*) Marquette Cement Mfg. Co., 140 So. Dearborn St., Chicago, Ill.

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Sieverling, P. A., (*1937*) Universal Atlas Cement Co., 630 Brown Marx Bldg., Birmingham, Ala.

***SIGNAL MOUNTAIN PORTLAND CEMENT CO.,** (*1936*) Volunteer Bldg., Chattanooga, Tenn. (I. F. Sisson)

Simonson, John C., (*1936*) P. O. Box 964, Oklahoma City, Okla.

Simpson, Geo., (*1915*) West Norwalk Road, Darien, Conn.

Sinclair, Daniel M., (*1939*) District Concrete Control Lab., New Bethlehem, Pa.

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Sinnickson, William B., (*1937*) 999 Westside Ave., Jersey City, N. J.

SISALKRAFT CO., THE, (*1929*) 205 West Wacker Dr., Chicago, Ill. (Franklin A. Richards)

Sisson, I. F. (See Signal Mountain Portland Cement Co.)

Skinner, Robert F., (*1936*) Bureau of Reclamation, Yakima, Wash.

Skramtajew, B. G., (*1936*) Manager, Concrete Laboratory of the Central Structural Institute of Moscow, Moscow, U. S. S. R.

Author: "Determining Concrete Strength for Control of Concrete in Structures" V. 34, p. 285.

Sloan, Francis P. (See Brennan & Sloan, Inc.)

Smaill, Wm., (*1925*) Northern Construction Co., Ltd., Vancouver Block, Vancouver, B. C., Canada

Small, M. H., (*1937*) 1019 Gloyd Bldg., Kansas City, Mo.

Smallhorn, E. R., (*1936*) Box 132, Hochelaga P. O., Montreal, Que., Canada

SMIDTH & CO., F. L., (*1925*) 225 Broadway, New York, N. Y. (Orla A. Larsen)

SMIDTH & CO. A/S, F. L., (*1935*) 35 Vestergade, Copenhagen, Denmark

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Smith, Albert, (See Smith & Brown, Engineers, Inc.)

Author: "Design of Concrete Buildings for Wind Stresses" V. 23, p. 101; "Reinforced Concrete Design Practice" V. 34, p. 465. Ch. Com., 306, "Permissible Openings in Construction," V. 26, p. 24; Ch. Com., 308, "Basis of Design for Hurricane Exposure," V. 27, p. 903.

Smith, Blaine S., (*1928*) Universal-Atlas Cement Co., 135 East 42nd St., New York, N. Y.

Smith, Blaine S. (See Universal Atlas Cement Co.)

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Smith, Major N. H., (*1922*) Imber Court, East Molesey, Surrey, England

Smith, R. A. B., (*1928*) Cement and Concrete Association, 52 Grosvenor Gardens, London, S. W. 1, England.

Smith, Walter M., (*1910*) Wisconsin University Library, Madison, Wis.

SMITH & BROWN, ENGINEERS, INC., (*1927*) 307 N. Michigan Ave., Chicago, Ill. (Albert Smith)

Smyth, Arthur P., (*1936*) c/o J. G. White Engrg. Corp., Port au Prince, Haiti, West Indies.

Snethlage, John B., (*1937*) 59 Maitland Ave., Hawthorne, N. J.

Snyder, Chas. A. (See Richmond Screw Anchor Co., Inc.)

Snyder, F. S., (*1936*) 400 Arsenal St., Watertown, Mass.

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SOCIETE ANONYME DES CHAUX ET CIMENTS DE LA FARGE ET DU TEIL (*1925*) Le Teil (Ardeche), France. (P. Lhopitallier)

SOCIETE DES CIMENTS FRANCAIS, (*1923*) 80 Rue Taitbout, Paris, France, (F. Turquais)

Sodemann, Franz, (*1934*) "Danalith," A/S, 3 Gyldenlovesgade, Copenhagen V. Denmark

Soldini, R. B., (See Monolith Portland Cement Co.)

SOLVAY PROCESS CO., THE (*1932*) Hopewell, Va. (T. C. Morris)

SOLVAY PROCESS CO., THE, (*1924*) Syracuse, N. Y.

Somers, Robert E., (*Student—1938*) 1132 S. 51st St., Philadelphia, Pa.

SOUTH DAKOTA CEMENT PLANT, (*1927*) Rapid City, S. Dak. (Wm. Fowden, Supt.)

SOUTHERN PACIFIC COMPANY, (*1937*) 65 Market St., Rm. 1049, San Francisco, Calif. (W. H. Kirkbride)

***SOUTHERN STATES PORTLAND CEMENT CO.,** (*1936*) Rockmart, Ga. (G. W. Jordan, Chief Chemist)

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SOUTHWESTERN PORTLAND CEMENT CO., (*1934*) Box 668, Osborn, Ohio (J. B. Alexander)

Spencer, Arthur C., (*1936*) Consolidated Gas Electric Light & Power Co., Room 315, Monument Street Bldg., Baltimore, Md.

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Author: "Measurement of the Moisture Content of Concrete" V. 34, p. 45.

Spencer, Samuel A., (*1937*) Box 2016, Sydenham, Port Elizabeth, So. Africa

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SPIEKER CO., THE HENRY J., (*1935*) Box 356, Toledo, Ohio (Karl C. Miller)

Spiker, W. C., (*1918*) W. C. Spiker & Co., 1209 35th St., N. W., Washington, D.C.

Spilkin, J. H., (*1938*) P. O. Box 7233, Johannesburg, So. Africa.

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Sprague, John C., (*1931*) Post Office Box 646, Huntington, W. Va.
Author: "Effect of Curing Temperature on the Compressive Strength of Concrete at Early Ages" V. 32, p. 212; "Evaluating Fines in Concrete on a Bleeding Test Basis" V. 33, p. 29.

Springer, G. P., (*1920*) Purdue University, West Lafayette, Ind.

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STANDARD LIME & STONE CO., (*1928*) Box 885, Martinsburg, W. Va. (Edward E. Driesbach)

STANDARD PORTLAND CEMENT CO., (*1930*) P. O. Box 110, Painesville, Ohio (Harry Stevens)

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Co-author (with D. E. Parsons) "Tests of Mesnager Hinges" V. 31, p. 304; (Wason Research Medal).

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Author: "Vibration of Concrete on San Francisco Oakland Bay Bridge" V. 31, p. 539; "Cement and Concrete Control, San Francisco Oakland Bay Bridge" V. 32, p. 1; (with Lester C. Meder) "Resistance of Cements to Attack by Sea Water and by Alakli Soils" V. 34, p. 433 (Wason Medal for Notable Research).

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Author: "Mass Concrete Research for Hoover Dam" V. 29, p. 305.

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Steiner, Alexander, (*1937*) Visegradi-utca 12 II. 2, Budapest, Hungary

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Author: "Isteg Steel for Concrete Reinforcement" V. 32, p. 183.

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Author: "The Economic Possibilities of Lightweight Aggregate in Building Construction" V. 16, p. 152. Ch. Com., 501, "Progress Report on Building Regulations for Reinforced Concrete" V. 31, p. 181; (tentative standard 1936); V. 32, p. 407. Member: Advisory Com., 1929-; Board of Direction, Director, 1934-35.

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Author: "The Cuban Hurricane" V. 23, p. 290; "Moment and Shear Diagrams for Continuous Beams and Rigid Frames" V. 26, p. 211.

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***STONE & WEBSTER ENGINEERING CORP.,** (*1920*) 49 Federal St., Boston, Mass. (E. A. Dockstader)

Stradley, Jr., Henry E., (*1937*) c/o U. S. Bureau of Reclamation, Cody, Wyo.

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STUART'S GRANOLITHIC CO., LTD., (*1929*) 101 Baker St., London, W. I., England. (Wm. Adams, Managing Director)

Stubbs, Jr., Frank W., (*1936*) Rhode Island State College, Kingston, R. I.

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***SUPERIOR CEMENT CORP.,** (*1936*) 306 Masonic Temple, Portsmouth, Ohio. (F. R. Kanengeiser, Pres.)

***SUPERIOR PORTLAND CEMENT, Inc.,** (*1936*) 1003 Seaboard Bldg., Seattle, Wash. (C. N. Reitze)

Sutherland, Hale, (*1916*) Lehigh University, Bethlehem, Pa.

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Author: (with D. A. Abrams) "Method of Testing Cement Pipe" V. 8, p. 713; (with H. F. Gonnerman) "Test of an Eight-year-old Flat-Slab Floor of the Western Newspaper Union Building" V. 14, p. 192; "Engineering Research in Concrete and Reinforced Concrete" V. 20, p. 168. Ch. Publications Com. 1914-15; Member: Publications Com., 1929-32, 2nd V. Pres., 1911-13; V. Pres., 1914-15. Awarded (1928) Turner Medal for "outstanding contributions to the knowledge of reinforced concrete design and construction."

Talbot, Kenneth H., (*1937*) c/o Harza Engrg. Corp., 27 Cumberland St., Charleston, S. C.

Author: "Methods of Constructing Concrete Houses" V. 14, p. 408.

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Taylor, Thomas G., (*1937*) Portland Cement Assn., 33 W. Grand Ave., Chicago, Ill.

Co-author: (with F. E. Richart and R. L. Brown) "The Effect of Plastic Flow in Rigid Frames of Reinforced Concrete" V. 30, p. 181; (with H. F. Gonnerman and A. G. Timms) "Effect of Calcium and Sodium Chlorides on Concrete When Used for Ice Removal" (progress report) V. 33, p. 107.

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TEXAS STATE HIGHWAY DEPT. (*1938*) Austin, Texas. (Julian Montgomery, St. Highway Engineer)

TEXAS TECHNOLOGICAL COLLEGE, (*1928*) Civil Engineering Dept., Lubbock, Tex. (J. H. Murdough)

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Author: "Concrete Aggregates" V. 2, p. 27; "Forms for Concrete Construction" V. 3, p. 64; "Report of Committee on Reinforced Concrete" V. 5, p. 454; "Longitudinal Reinforcement in Concrete Columns" V. 6, p. 115; "The Practical Design of Reinforced Concrete Flat Slabs" V. 8, p. 254; (with Edward Smulski) "Design of Rigid Frames in Steel and Reinforced Concrete" V. 9, p. 156; "Design and Construction of the Massachusetts Institute of Technology Buildings" V. 11, p. 377; "Unit Costs in Construction" V. 12, p. 347; "Slag as a Concrete Aggregate" V. 13, p. 107; Sectional

V. P., Reinforced Concrete, 1908; Sectional V. P., Specifications and Methods of Tests for Concrete Materials, 1911-13. Ch., "Report of Committee on Specifications and Methods of Tests for Concrete" V. 8, p. 473; V. 9, p. 356; V. 10, p. 422; V. 12, p. 478; Ch. Com., "Concrete Aggregates," V. 13, p. 313; V. 16, p. 230. V. Pres., 1917-20.

Thompson, Warren H., (*1935*) c/o Buffalo Steel Co., Tonawanda, N. Y.

THOMPSON & LICHTNER CO. INC., (*1925*) 620 Newbury St., Boston, Mass. (Miles N. Clair)

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Author: "The Design and Operation of Central Mixing Plants" V. 26, p. 551. Member: Board of Direction, 1939-.

Thornton, William C., (*1937*) U. S. Engineer Office, Providence, R. I.

Thorvaldson, T., (*1927*) University of Saskatchewan, Saskatoon, Canada
Author: (with D. Wolochow) "The Action of Sulphate Solutions on Steam-Cured Composite Cement Mortars" V. 34, p. 241.

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Timms, A. G., (*1938*) 33 West Grand Ave., Chicago, Ill.
Co-author: (with R. E. Copeland) "Effect of Mortar Strength and Strength of Unit on the Strength of Concrete Masonry Walls" V. 28, p. 551; (with N. H. Withey) "Temperature Effects on Compressive Strength of Concrete" V. 30, p. 159; (Wason Research Medal); (with N. H. Withey) "Further Studies of Temperature Effects on Compressive Strength of Concrete" V. 31, p. 165; (with T. G. Taylor and H. E. Gonnerman) "Effect of Calcium and Sodium Chlorides on Concrete When Used for Ice Removal" (Progress report) V. 33, p. 107.

Titsworth, H. H. (See Glens Falls Portland Cement Co.)

Toch, Maximilian, (*1906*) c/o Toch Bros., 50 Central Park W., New York, N. Y.
Author: "Shall Anything be Added to Portland Cement" V. 21, p. 134.

Todd, G. H., (*1937*) Texas Testing Laboratories, Inc., 1327 Wood St., Dallas, Texas

TORONTO, DEPT. OF BUILDINGS, (*1934*) City Hall, Toronto, Ont., Canada, (K. S. Gillies)

Torres, Ary F., (*1927*) Institute de Pasquisas Technologicas Do Est. de S. Paulo, Sao Paulo, Brazil

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Author: "Advantages in the Use of High Strength Concretes" V. 28, p. 607.

Tozzer, A. C. (See Turner Construction Co.)
Author: "Concrete Work on the Brooklyn Army Base" V. 15, p. 139. Member: Board of Direction, Director, 1926-28.

Trainor, Lee S., (*1933*) National Lime Association, 927 Fifteenth St. N. W., Washington, D. C.

Travis, W. (See Industrial Engineering Co.)

Traynor, F. M. (See Florida Portland Cement Co.)

Tremper, Bailey (See Washington State Highway Dept.)
Author: "The Effect of Acid Waters on Concrete" V. 28, p. 1.

Trice, A. C., (*1938*) 2600 Cumberland Ave., Middlesboro, Ky.

***TRINITY PORTLAND CEMENT CO.,** (*1936*) Republic Bank Bldg., Dallas, Texas (J. F. Hayden)

Trotter, C. T., (*1928*) Standard Clay Products Ltd., St. Johns, P. Q., Canada

Troupp, N., (*1934*) Pargas Kalkbergs Aktiebolag, Pargas, Finland

Troxell, G. E., (*1929*) Assoc. Prof. of Civil Engineering, University of California, Berkeley, Calif.
Co-author: (with Raymond E. Davis) "Volumetric Changes in Portland Cement Mortars and Concretes" V. 25, p. 210; (with R. E. Davis) "Properties of Mass Con-

crete" (Com. 108) V. 27, p. 385; (with R. E. Davis, R. W. Carlson and J. W. Kelly) "Cement Investigations for the Hoover Dam" V. 29, p. 413; (with R. E. Davis, R. W. Carlson and J. W. Kelly) "Properties of Mortars and Concretes Containing High Silica Cements" V. 30, p. 369; (with R. E. Davis, R. W. Carlson and J. W. Kelly) "Cement Investigations for Boulder Dam with the Results up to the Age of One Year" V. 30, p. 485 (Wason Medal for most meritorious paper); (with R. E. Davis, J. W. Kelly and Harmer E. Davis) "Properties of Mortars and Concretes Containing Portland-puzzolan Cements" V. 32, p. 80; Author: "A Short-Time Test for Effect of Type of Cement on Concrete Shrinkage" V. 35, p. 73. Ch. Com., 102, "Proper Methods of Design and Construction of Concrete Structures to Prevent Damage from Volumetric Changes of the Concrete" V. 30, p. 209.

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Tso, Meng Hsing (*Student—1939*) c/o Mr. Ching-Hai Li, 125 Dryden Road, Ithaca, N. Y.

Tubesing, Wm. F., (*1937*) 8011 Chestnut St., Wauwatosa, Wisc.

Tucker, C. Sheldon (*1939*) 642 Seybold Bldg., Miami, Fla.

Tucker, Jr., John (*1926*) 5403 39th St. N. W., Washington, D. C.
Author: (with G. W. Walker) "The Physical Properties of Commercial Cast Stone" V. 25, p. 501; (with G. W. Walker and J. Arthur Swenson) "The Physical Properties of Cast Stone" V. 28, p. 243.

Turneaure, F. E., (*1932—Honorary*) 166 N. Prospect Ave., Madison, Wis.
Awarded (1930) Turner Medal for "Distinguished service in formulating sound principles of reinforced concrete design."

Turner, Henry C., (*1905—Honorary*) Turner Construction Co., 420 Lexington Ave., New York, N. Y.
Author: "Aims and Activities of the American Concrete Institute" V. 17, p. 20. Ch. Com., Reinforced Concrete Barges and Ships, V. 14, p. 505; V. 15, p. 312; V. 16, p. 161. Member: Board of Direction, Director, 1914-15; Treasurer, 1906-13; V. Pres., 1915-19; Pres. 1920-21; Past Pres., 1922-31. Founder (1927) of the Henry C. Turner Award (gold medal) "for notable achievement in or service to the concrete industry."

***TURNER CONSTRUCTION CO.,** (*1919*) Architects' Bldg., Philadelphia, Pa. (H. C. Turner, Jr.)

***TURNER CONSTRUCTION CO.,** (*1920*) 420 Lexington Ave., New York, N. Y. (A. C. Tozzer)

Turner, Jr., H. C. (See Turner Construction Co.)

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Author: "Vibration as an Aid in Placing Better Concrete" V. 31, p. 545; "Resistance of Cement to the Corrosive Action of Sodium Sulphate Solutions" V. 33, p. 83.; (with Harmer E. Davis) "Overvibration and Revibration of Concrete" V. 35, p. 41.

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Author: "Concrete at Norris Dam" V. 32, p. 295.

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Udani, P. J., (*1937*) Bharti Society, Ellis Bridge, Ahmedabad, India

ULTIMATE PAVING BLOCK CORP. OF AMERICA, (*1937*) 141 Broadway, New York, N. Y. (Louis F. Hewett)

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Unger, Robert J., (*1939*) 36 Derby Ave., New Haven, Conn.

UNITED CARBON CO. INC., (*1937*) Charleston, W. Va. (G. L. Roberts)

UNITED ENGINEERS & CONSTRUCTORS, INC., (*1929*) 1401 Arch St., Philadelphia, Pa. (J. Wm Moffet)

***UNIVERSAL ATLAS CEMENT CO.,** (*1914*) 210 So. LaSalle St., Chicago, Ill. (Blaine S. Smith)

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Author: "Time as a Factor in Making Concrete" V. 23, p. 434; "Concrete for Foundations and Water-Front Work" V. 20, p. 135; "President's Address" V. 23, p. 22; "President's Address" V. 24, p. 21; "Recent Developments in Pile Foundations" V. 33, p. 557. Ch. Com., E-6, Destructive Agents and Protective Treatments, V. 19, p. 301; V. 20, p. 564; V. 21, p. 266; V. 22, p. 641. V. Pres., 1922-25; Pres., 1926-27; Past Pres., 1928-1936.

Upton, L. (See Bound Brook Crushed Stone Co.)

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Walker, Stanton (See National Sand & Gravel Assn.)

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Arizona

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Spiker, W. C.
Stang, Ambrose H.
Stathis, Nicholas P.
Steele, Byram W.
Stephens, Clifford S.
Trainor, Lee S.
Tucker, Jr., John
Walker, Stanton
Yingling, Clinton K.

Florida

Gainesville—Miltimore, Cora
Miami—Hardin, Hal P.
Tucker, C. Sheldon
Tampa—Brown, Geo. D.
Florida Portland Cement Co. (F. M. Traynor)

Georgia

Atlanta—Beer, Robert G.
Crocker, C. N.
Gash, Tom
Humphries, H. D.
McDonald, J. Harry
Oliver, Herbert D.
Dalton—Marshall, Jr., J. M.
Rockmart—Southern States Portland Cement Co. (G. W. Jordan)
Savannah—Charms, Richard de

Idaho

Boise—Newell, Robert J.
Sanford, H.
Caldwell—Higginson, Elmo C.
Twin Springs—Studebaker, Claude H.

Illinois

Champaign—Kent, Paul F.
Roy, N. H.
Chicago—Affleck, B. F.
Alexander, E. C.
Allan, W. D. M.
Allen, O. T.
American Concrete Pipe Assn. (M. W. Loving)
Arndt, Alfred F.
Battey, Paul L.
Beinhauer, Frank H.
Boase, A. J.
Boettner, J. L.
Bowlby, H. L.
Brundage, Avery
Bullen, C. H.
Chicago University of, (L. R. Flook)
Cleary, James M.
Concrete Reinforcing Steel Inst. (Mark Beeman)
Cowham Engineering Co. (A. E. Hjerpe)
Cummings, A. E.
Davis, G. E.
Duffy, Joseph L.
Emerson, H. B.
Epstein, A.
Flood, Walter H.
Forbrich, Louis R.
Gibson, E. L.
Gonnerman, H. F.
Gosswein, O. H.
Grinter, L. E.
Heckel, E. J.
Hill, W. N.
Hunter, Robert C.
Huntly, Phil C.
Hunt Co., Robert W.
Inland Steel Co. (H. H. Straus)
Kaiser, W. G.
Kalinka, J. E.
Kanter, J. A.
Karcz, Michael J.
Keatinge, Paul F.
Kinney, William M.
Konstant, Nicholas Z.
Kushing, John W.
Leffler, Ralph R.
Lehle, George L.
Lindsay, G. L.
Logeman, R. T.
Lord, Arthur R.
Lurie, E. M.
McClurg, Verne O.
McKeague, E. D.
McKenzie, N. H.
McMillan, Franklin R.
Mall Tool Company (M. Rehnquist)
Many, Ben J.
Marden, E. R.
Marquette Cement Mfg. Co. (D. S. Colburn)
Marquette Cement Mfg. Co. (W. A. Wecker, Pres.)
Masonite Corporation (Thos. D. Gaskin)
Menzel, Carl A.
Molke, Eric C.
Moulton, A. G.
Palmieri, Mario
Pearse, Langdon
Peck, Roy L.
Phillips, R. S.
Post, C. L.
Powers, Treval C.
Rader, B. H.
Rail Steel Bar Assn. (H. P. Bigler)
Ramseyer, Chas. F.

Randall, Frank A.
Rockwood, Nathan C.
Rogers, Lester C.
Sauermann, Otto
Schneider, H. B.
Schuettge, Ralph T.
Scott, Robert G.
Sheets, Frank T.
Shields, H. C.
Siess, Chester P.
Sisalkraft Co., The (Franklin A. Richards)
Smikel, Frank H.
Smith & Brown, Engineers, Inc. (Albert Smith)
Stedje, J. J.
Stineman, Norman M.
Taylor, Thomas G.
Timms, A. G.
Universal Atlas Cement Co. (Blaine S. Smith)
Universal Atlas Cement Co. (F. L. Stone)
Van Trump, Isaac
Walker, C. G.
Walker, Frank R.
Webb, Claude A.
Wells, F. Harris
Wertz, Louis S.
Witt, J. C.
East St. Louis—Ayers, L. K.
Evanston—Boulton, H. S.
Lindau, A. E.
Maney, G. A.
Forest Park—Kahn, Asher D.
Glen Ellyn—Collier, Theodore F.
Copeland, R. E.
Morgan, Charles E.
Hinsdale—Brock, Arthur S.
La Salle—Butler, C. M.
Moline—Keller, Norman E.
Oglesby—Groll, Philip A.
Rockford—Ipsen, Mogens
Springfield—Glover, V. L.
Philbrook, L. E.
Whannel, R. L.
Urbana—Bauer, E. E.
Brielmaier, A. A.
Crandell, John S.
Dell, George H.
Ehasz F. L.
Fink, George R.
Huntington, W. C.
Jensen, V. P.
Kluge, Ralph W.
Morgan, N. D.
Newmark, Nathan M.
Penman, Robert R.
Renner, W. A.
Richart, F. E.
Shedd, Thomas C.
Talbot, Arthur N.
Wilson, Wilbur M.
Zaborowski, Robert
Western Springs—Schad, James A.

Indiana

Bloomfield—Brassert, Walter
East Chicago—Gaither, J. T.
Fort Wayne—Droege, William H.
Miller, J. E.
Hammond—Besozzi, Leo
Indianapolis—Baker, Hugh J.
Hindman, R. B.
Rayburn, Jr., E. B.
Rohlwing, A. W.
Lafayette—Crepps, R. B.
Hatt. William K.
Hubler, J. W.
Mills, R. E.
Poorman, A. P.
Springer, G. P.
Speed—Louisville Cement Co. (H. D. Baylor)

Iowa

Ames—Beal, Robert W.
Blumenschein, E. W.
Caughey, Robert A.
Dunagan, W. M.
Gilkey, Herbert J.
Kerekes, Frank
Morris, Mark
Murphy, Glenn
Stoddard, A. E.
Davenport—Comess, Sam
Crabbs, Austin
Ferrel, K. P.

Des Moines—Dierking, R. F.
Fisher, R. J.
Handy, Walter N.
Mandia, Jas. V.
Patzig, Monroe L.
Iowa City—Posey, Chesley J.
Iowa Falls—Rossiter, Paul
Welden, E.
Mason City—Northwestern States Portland Cement Co. (G. C. Blackmore)
Northwestern States Portland Cement Co. (G. C. Blackmore)
Moorhead—Jones, Ray W.
Moville—Everhart, C. C.
Sioux City—Heitman, Richard
Johnson, T. H.
Waterloo—Bluedorn, Carl A.
Douthett, C. L.

Kansas

Chanute—Ash Grove Lime & Portland Cement Co. (A. D. Conrow)
Gardner, Guy O.
Fredonia—Dennis, J. W.
Piller, A. Edward
Fort Scott—Thomas, Howard M.
Hutchinson—Grothe, Wm. F.
Manhattan—McCaulley, George
Topeka—Justice, R. J.
Pierce, A. F.

Kentucky

Bowling Green—Jones, Clarence W.
Frankfort—Anderson II, F. Paul
Louisville—American Builders Supply Co. (Jas. F. McCracken)
Kosmos Portland Cement Co. (A. W. Horner)
Middlesboro—Trice, A. C.

Louisiana

Monroe—Neff, Stewart S.
New Orleans—Derickson, Donald
Lone Star Cement Corp. (H. A. Sawyer)
Rice, George P.
Shilstone, Herbert M.
Ruston—Sessums, Roy T.
University—Bateman, John Henry

Maine

Augusta—Mason, Frank H.
Orono—Technology Experiment Station, University of Maine (H. Walter Leavitt)

Maryland

Baltimore—Greiner, J. E.
Higgins, N. B.
Hogan, J. V.
Lane, H. A.
Spencer, Arthur C.
Timlin, M. A.
Bethesda—Brouk, Otto
Carderock—Fischer, Hugo C.
Chevy Chase—Herschel, Winslow H.
College Park—Ernst, George C.
Longwoods—Lockhart, 3rd, Henry
Union Bridge—LaForge, G. S.

Massachusetts

Belmont—Brickett, Edward M.
Garland, Chesley F.
Boston—Allbright, Edwin F.
Alvord, Henry B.
Amirian, Thomas T.
Blodgett, Malcolm
Bratt, Albert V.
Brown, Maurice F.
Chellis, Robert D.
Cheever, Walter G.
Corson, R. C.
Cowan, Benjamin M.
Cundari, Frank A.
Cutler, M. H.
Dobler, Oscar S.
Dockstader, E. A.
Fay, Frederick H.
Jackson & Moreland (F. M. Carhart)
Linenthal, Mark
McMullen, Ernst W.
Main, Charles T.
Nichols, John R.
Norton, Paul W.

Norwood, Edgar A.
Reidy, Maurice A.
Ryan, Charles N.
Sherman, Herbert L.
Stone & Webster Engrg. Corp. (E. A. Dockstader)
Thompson, Sanford E.
Thompson-Lichtner Co. Inc. (Miles N. Clair)
Turner, J. M.
Worcester, J. R.
Brighton—McCarthy, Robert E.
Cambridge—Carlson, Roy W.
Dahanukar, S. M.
Fife, W. M.
Haertlein, Albert
Kennedy, Henry L.
Mehringer, Frank J.
Mirabelli, Eugene
Mitsch, John D.
Peabody, Jr., Dean
Rappoli, Edmund J.
Reading, Thomas J.
Staley, Howard R.
Voss, Walter C.
Westergaard, Harald M.
Wilson, Francis W.
Clinton—Innamorati, A. W.
Dedham—Bjornson, Henry L.
Dorchester—Gibson, Arthur E.
O'Rourke, James A.
Everett—McGillivray, Harold Joseph
Lenoxdale—Berkshire Gravel Co. (A. I. Newton)
Lynn—Hilton, William B.
Medford—Allen, Cleveland
Freedman, J.
Mardulier, F. J.
Newton, Donald
Parks, Frederick B.
Powers, Chester J.
Melrose—Cates, Vincent K.
Milford—Corbett, W. E.
Newton—Bender, E. O.
Sampson, George A.
Newton Center—Phillips, Silas B.
Somerville—Gray, Howard Allison
Hurley, John J.
Springfield—Roy, Joseph E.
Swampscott—Hume Pipe of New England, Inc. (Joseph A. Dunn)
Latham, Charles H.
Taunton—Rose, Joseph S.
Watertown—Snyder, F. S.
Wellesley—Markey, Joseph J.
Protze, Jr., Herman G.
Winchester—Allen, John E.
Ball, Donald G.

Michigan

Alpena—Besser Manufacturing Co. (J. H. Besser, Pres.)
Boyd, Edwin A.
Ann Arbor—Emmons, W. J.
Menefee, F. N.
Sherlock, R. H.
Coldwater—Wolverine Portland Cement Co.
Detroit—Brown, W. Durand
Calcium Chloride Assn. (Fred Burggraf)
Cooper, Gage W.
Detroit Edison Co. (A. S. Douglass)
Elms, Stephen F.
Hirshfeld, C. F.
Huron Portland Cement Co. (A. J. Rooney, V. P.)
Johnston, C. C.
Kahn, Albert
Kennedy, John W.
Lenhardt, L. G.
MacKenzie, W. S.
Maguire, Walter J.
Mason, Roscoe J.
Metzen, E. B.
Morse, Floyd C.
Nelles, John S.
O'Connor, Albert J.
Peerless Cement Corp. (Charles A. Luck)
Peerless Cement Corp. (H. L. Henson)
Peerless Cement Corp. (A. C. Eichenlaub)
Perry, Ralph W.
Reed-Lewis, E. W.
Sexton, James E.

Strohm, Adam
Wayne University (Dudley Newton)
Whipple, Harvey
Wilk, Benjamin
Fenton—Hiscox, A. E.
Grand Rapids—Seeger, Ralph E.
Jackson—Consolidated Cement Corp.
Lansing—Davis, A. M.
La Camera, Peter
McLaughlin, William W.
Ludington — Electric Tamper & Equipment Co.
Midland—Dow Chemical Co. (L. C. Stewart)
Oxford—Williams, L. E.
Quincy—Deane II, Frank Putnam

Minnesota

Duluth—Boswell, Clay C.
LaLiberte, Paul
Polaris Concrete Products Co. (Gordon H. Butler)
Elk River—Longfellow, D. W.
Eveleth—Sholund, H. D.
Minneapolis—Altman, Frank S.
Cheng, T. P.
Clousing, Louis
Hughes, C. A.
Klassy, K. H.
Krefting, A. S.
Macgowan, Ernest S.
Moore, O. F.
Paul, Frederick T.
Peterson, F. G. Eric
Straub, Dr. Lorenz G.
Wheeler, Walter H.
Wise, Joseph A.
St. Paul—Britzius, C. W.
Bryce, Arthur J.
Darling, Wilfred D.
Gorham, Fred A.
Miller, Dalton G.
Schoell, William D.
Schuett, A. H.
Shockey, Norvell A.
Wilbur, C. C.
Winona—Gurney, Wm. G.

Mississippi

Jackson—Kinnaird, Jr., Robert N.
Wier, G. W.
Wood, Robert H.
State College—McCain, D. M.

Missouri

Clarksville—Ivie, William B.
Columbia—Moorman, Robert W. B.
Jefferson City—Missouri State Highway Dept. (F. V. Reagel)
Kansas City—Allen, H. S.
American Aggregate Co. (J. B. Cleary)
Black & Veatch (N. T. Veatch Jr.)
Cleary, John B.
Cross, Roy C.
Dewey Portland Cement Co. (W. E. Tyler)
Dewey Portland Cement Co. (W. E. Tyler)
Doll, Theodore
Lundteigen, A.
Porter, G. W.
Small, M. H.
Timanus, C. S.
Wilkes, Edmund
St. Louis—Becker, W. C. E.
Beyer, E. L.
Deacon, William T.
Downing, C. A.
Fleming, T. H.
Frech, Harry E.
Friberg, Bengt.
Koerner, Carl A.
Laclede Steel Co. (B. Friberg)
Middleton, Harvey
Missouri Portland Cement Co.
Parcel, John I.
Pettus, L. A.
Rodgers, Eden
Sweetser, Ernest O.
Thomson, H. F.
Walker, Otis H.
Widmer, Arthur J.

Montana

Fairfield—Genger, Harold W.
Glendive—Jones, Paul A.
Havre—Callan, James A.
Johnston, Harold A.

Nebraska

Lincoln—Duff, C. M.
Sampson, B.
Louisville—Frolich, A. K.
Omaha—Meyer, C. Louis
Yant, R. S.

New Hampshire

Portsmouth—Cotter, C. H.
Tilton—Mayer, Frank F.

New Jersey

Ampere—Lock Joint Pipe Co. (A. M. Hirsh)
Arlington—Walter, Ernest
Bound Brook—Lloyd, H. E.
Burlington—Hunt, Horace S.
Camden—Camden Lime Company (Frank S. VanHart)
Demarest—Abberley, Elbert K.
Dunellen—Ransome Concrete Machinery Co. (L. R. Wilson)
East Orange—Dunham, Clarence W.
East Rutherford—Wright, Jr., Wm. R.
Elizabeth—Kropp, Robert
Hackensack—LaFountain, A. A.
Haddonfield—Kolyn, M. D.
Hawthorne—Snethlage, John B.
Hoboken—Hirschthal, Meyer
Vogel, John Leonard
Jersey City—Sinnickson, William B.
Walter, L. W.
Manasquan—Brandes, Raymond L.
Manville—Rembert, E. Wayne
Maplewood—Hunt, Gilbert A.
Newark—Anti-Hydro Waterproofing Co. (M. W. Meyer)
Bound Brook Crushed Stone Co. (L. Upton)
Johnson, Nathan C.
Powers, Ellwood D.
Shaw, Saul
Waldron, William
New Brunswick—Kauffmann, John R.
Marin, Joseph
New Village—Wilsnack, George C.
Orange—Mead, F. J.
Phillipsburg—Fritts, Stewart S.
Plainfield—Wigton, C. B.
Princeton—Timby, Elmer K.
Tschebotareff, Gregory P.
Rutherford—Fink, John C.
Trenton—Andrews, L. E.
Gage, Robert B.
Johannesson, Sigvald
Scheirer, M. K.
Sherman, Ralph A.
West Englewood—Paliska, Stephen G.
Westfield—Grove, William G.

New Mexico

Conchas Dam—Johnson, Ralph P.
Rippon, Charles S.

New York

Albany—Greenman, Russell S.
McClure, James
Paul, Ira
Schultze, Paul
Arkport—Wright, Leslie E.
Binghamton—Scholer, Charles H.
Brooklyn—Berg, U. T.
Central Concrete, Inc. (A. Johnson, Treas.)
Kaspareit, Walter B.
Moore, Roy Saxton
Praeger, Emil
Richmond Screw Anchor Co., Inc. (Chas. A. Snyder, Pres.)
Roux, Maurice G.
Ryan Co., N. (M. C. Golden)
Buffalo—Buffalo Slag Co. (Carlton S. Wicker)
Clark, W. B.
Cowper, John W.
Creager, Wm. Pitcher
Cruise, Richard R.
Feigel, John H.
Fruchtbaum, J.
Gill, James F.

Hallenbeck, Geo. S.
Hudson, William N.
Knight, T. C.
Locke, Clyde E.
McKaig, Thomas H.
Niagara Falls Power Co.
Stelley, Harry A.
Waters, Charles R.
West, Wilbur T.
Catskill—North American Cement Corp. (H. F. Kichline)
Cattaraugus—Babcock, Charles D.
East Aurora—Watson, T. P.
Elmira—Patterson, Donald M.
Flushing—Lemcke, Karl W.
Glens Falls—Glens Falls Portland Cement Co. (H. H. Titsworth)
Raber, Chas. T.
Greensboro—Blakeley, Harry E.
Hornell—Nichols, T. F.
Hudson—Lone Star Cement Corp. (Myron A. Swayze)
Ithaca—Hollister, S. C.
Huang, C.
International Salt Co., Inc. (C. D. Looker)
O'Rourke, Chas. E.
Schickel, Norbert H.
Scofield, H. H.
Tso, Meng Hsing
Urquhart, L. C.
Long Island City—Horn, A. E.
Levine, Harold J.
Astoria, L. I.—Polivka, Jaroslav
Westbury, L. I.—Piloff, Albert K.
New Rochelle—Steljes, Martin
New York—Abrams, Duff A.
Ahlers, John G.
Ahrens, Henry
Ambursen Construction Co. Inc. (L. A. Robb)
American Can Co. (C. E. Preis)
American Isteg Steel Corp. (J. Mercadante)
Backus, Richard A.
Barney Corp., W. J. (Wm. Joshua Barney)
Bertin, Rene L.
Billner, K. P.
Borchard, E. K.
Borsari Tank Corp. of America (Herman Schorer)
Brennan & Sloan, Inc. (Francis P. Sloan)
Caiola, Fred
Chapman, Cloyd M.
Cohen, A. B.
Concrete Steel Co. (John F. Havemeyer)
Connar, V. N.
Cory, Russell G.
Crawford, Gordon L.
Cremer, Randall
Creskoff, Jacob J.
Crosett, Alexander D.
Deutschbein, H. J.
Di Stasio, Joseph
Dixey, Joseph H.
Dixon, D. H.
Farmer, Homer G.
Feldman, Solomon G.
Fischer, Freidrich
Fleming, E. M.
Ford, Bacon & Davis, Inc. (Geo. I. Rhodes)
Francisco, F. LeRoy
Freeman, G. L.
Galloway, T. R.
Gannett, J. K.
Giles, Roy T.
Gilman, Charles
Gruenwald, Ernst
Haller, R. T.
Hardesty, J. M.
Hayden, Harold P.
Industrial Engineering Co. (W. Travis)
Josephs, Arthur C.
Kelly, Frank L.
Kessler, Matthew J.
Kinneman, W. P.
Krefeld, William J.
Knochenhauer, Theodore G.
Lawrence Portland Cement Co. (E. P. Haubert)
Leftwich, R. F.
Litter, F. J.
Liu, H. S.

Lockhardt, William F.
Lone Star Cement Corp. (H. R. Durbin)
Lone Star Cement Corp. (C. L. Hogan, Pres.)
Lone Star Cement Corp. (R. A. Hummel, V. P.
Lone Star Cement Corp. (E. Posselt, V. P.)
Lone Star Cement Corp. (Dewey Thomspon)
Manhattan College Engineering School (Prof. Daniel J. O'Connell)
Maurer, E. R.
Merriman, Thaddeus
Meyers, Abraham
Miner, Joshua L.
Moore, O. L.
Moyer, Albert
Mueser, William H.
Murphy, James F
North American Cement Corp.
N. Y. C. Dept. of Public Works (Wm. S. Elliott)
Pacanins, Tomas
Peck, John Sanford
Pennsylvania - Dixie Cement Corp. (Joseph H. Chubb)
Pitman, E. P.
Prentice, Thos. H.
Raymond Concrete Pile Co. (H. P. Hamlin)
Republic Fireproofing Co., Inc. (Alden J. Palmer)
Reynolds, Frank L.
Richard, N. A.
Ridgway, John J.
Roberts, H. W.
Rogers, Ansel T.
Rose, W. A.
Scheinin, Joseph M.
Schmitt, F. E.
Seabury, George T.
Searl, Thomas D.
Shreve, R. H.
Smidth & Co., F. L. (Orla A. Larsen)
Smith, Blaine S.
Smith, Kenneth A.
Sporn, Philip
Steinman, D. B.
Stephens, A. W.
Stites, J. D.
Toch, Maximilian
Turner, Henry C.
Turner Construction Co. (A. C. Tozzer)
Ultimite Paving Block Corp. of America (Louis F. Hewett)
Upson, Maxwell M.
Van Buren, Maurice P.
Vinci, Alfonso
Wait, Bertrand H.
Wessman, Harold E.
White Construction Co., Inc. (G. Edward Escher)
Wickwire Spencer Steel Co. (R. L. Foster)
Wilcox, Fred C.
Willison, M. W.
Woodard, Silas H.
Ytterberg, C. F.
Zipprodt, Roy R.

Niagara Falls—Kremers, Ernest

Rochester—Brookman, Roger S.
Campbell, S. A.
Davis, Frank R.
Domine, H. P.
Hughes, H. Walter
Klock, Morgan B.
Palmer, B. H.
Rochester Gas & Electric Corp. (Walter E. Paine)
Sanford, L. M.
Scherer, Francis R.
Spencer, William H.

Rockville Center—Moore, A. F.

Schnectady—Hall, Russell A.

Snyder—Vannier, C. B.

Syracuse—Solvay Process Co., The

Tonawanda—Thompson, Warren H.

Valhalla—Riddle, C. D.

Webster—Sheridan, Robert R.

West Point—Smith, Morris R.
Wuerpel, Chas. E.

White Plains—Hepburn, William

Yonkers—Hernandez, Clinton N.
Keaveny, Felix J.

North Carolina

Asheville—Rawls, E. M.
Charlotte—Wanzer, C. T.
Greensboro—Arnold, M. A.
Hiwassee Dam—McCluer, Sam
Raleigh—Hutchinson, G. W.

Ohio

Akron—Davis, Marvin L.
Cincinnati—Christy, C. O.
Duvall, Belmon V.
Emery, S. A.
Faber, H. A.
Hanly, Hunter W.
Little, Charles R.
Long, Bartlett Gaillard
Maienschein, Joseph E.
Meyer, Geo. W.
Olah, Joseph J.
Pollak, Julian A.
Renz, Carl F.
Van Camp, R. E.
Warner, J. J.
Cleveland—Barnes, George E.
Burger, Alfred A.
Churchill, H. D.
Deinboll, F. K.
Falconer, R. C.
Flesheim, S. W.
Herron, James H.
Hoffman, Oscar
Kaufman, R. R.
Ketchum, Jr., Milo S.
Klapp, Frank O.
Low, Albert S.
Maginniss, H. J.
Medusa Portland Cement Co.
Mitchell, Thomas H.
Mooney, Philip
Pease, B. S.
Plummer, Fred L.
Thwing, James
Watson, Wilbur J.
Williams, C. D.
Columbus—Cummins, E. W.
Elford, H.
Jaeger Machine Co. (Lion Gardiner)
Large, George E.
Morris, Clyde T.
Shank, J. R.
Stepanian, S.
Sterner, Howard S.
Unckrich, A. F.
Weinland, Jr., Louis A.
Dayton—Belz, Charles J.
Kimmel, A. W.
McCrery, H. E.
Lakewood—Feldrappe, M. G.
Lucasville—Goodman, Hal W.
Osborn—Southwestern Portland Cement Co. (J. B. Alexander)
Painesville—Standard Portland Cement Co. (Harry Stevens)
Portsmouth—Superior Cement Corp. (F. R. Kanengeiser)
Shaker Heights—Scripture, Jr., Edward W.
South Park—Barney, W. E.
Toledo—Carlisle, W. C.
Hausman, Isaac
Loughlin, L. J.
Nardiello, M. V.
Reese, Raymond C.
The Henry J. Spieker Co. (Karl C. Miller)
Youngstown—Hubbard, Fred
Roscoe, Ralph E.
Stewart, W. H.

Oklahoma

Dewey—Chamberlain, P. R.
Oklahoma City—Oakes, Herbert L.
Simonson, John C.
Smith, W. Furber
Winters, R. W.
Vinita—Carpenter, Hubbell
Fuller, M. G.

Oregon

Bend—Knight, Elton G.
Portland—Blake, Harold
Brown, Jr., Harlan E.
Clark, R. R.
Cooper, Miles K.
McCullough, Conde B.
Nims, Chas. B.
Oregon Portland Cement Co. (D. H. Leche, Gen. Supt.)
Salem—Paxson, G. S.

Pennsylvania

Allentown—Collins, H. G.
Lehigh Portland Cement Co.
Lehigh Portland Cement Co.
Lehigh Portland Cement Co.
Pearson, J. C.
Watt, A. G.
Wise, Cooper S.
Young, Roy N.
Ambler—Keasbey & Mattison Co. (C. R. Hutchcroft)
Ardmore—Erickson, Roy J. A.
Bethlehem—Bethlehem Steel Corp. (A. P. Clark)
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Braddock—McCrady, Jr., Edward
Butler—West Penn Cement Co. (O. J. Binford)
Catasauqua — Allentown Portland Cement Co. (R. S. Weaver)
Allentown Portland Cement Co. (F. A. Weibel)
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Whitehall Cement Mfg. Co. (M. H. Meighan, Ch. Chemist)
Coplay—Gruenwald, Max E.
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Gettysburg—Clutz, Frank H.
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Harrisburg—Craig, Robert Hall
Flesher, Wm.
Masters, F. M.
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Hummelstown—Jones, William A.
Johnstown—Hertz, A. L.
Narrow, Israel
Lancaster—Wickersham, John H.
Lewisburg—Martin, Eliza J.
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Nazareth Cement Co. (J. Clifford Evans)
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Torres, Ary F.

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England

Bennetthorpe—Taylor, Maurice
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Davis, A. C.
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Stanger, R. H. Harry
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Windsor, A. S.
Manchester—Stokes, G. W.
Oxford—Jackson, Alan
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Tunbridge Wells—Bishop, Hugh P.
Wolverhampton—Cotterell, Ernest L.

Esthonia

Tallinn—Grauen, A.

Finland

Pargas—Troupp, N.

France

Le Teil—Societe Anonyme des Chaux et Ciments de Lafarge et du Teil (P. Lhopitallier)
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Laboratories du Batiment et des Travaux Publics
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PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE

VOLUME 35—1939

From JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, Vol. 10, Sept. 1938, to June 1939 and Supplement, Sept. 1939

This is an Index of:

Original contributions to the JOURNAL OF THE AMERICAN CONCRETE INSTITUTE—papers, reports, discussions by subject, title and author.

For the convenience of the reader who is referring to JOURNAL issues rather than to Bound Volumes (in which discussion is assembled following the paper discussed) there is a reference to JOURNAL issue in which discussion appeared. This reference is by supplementary page numbers. If the last page of a paper is 24, the first page of the discussion, published later, is 24-1.

In general, important subjects are classified and indexed under approximately 30 main headings each one appearing in its proper alphabetical order in bold face capital letters—as for instance, ARCHITECTURAL DESIGN. "Tower," and other subjects classified under this head, are indented.

Key words to important subjects, appear, in alphabetical order in addition to the general classification—as for instance "Admixtures" and "Aggregates" each referring to MATERIALS AND TESTS under which all allied references appear, indented. Authors' names and original titles of papers appear in bold face type in proper alphabetical sequence with the subjects, with references to their contributions.

Specific data on Beams are so indexed by reference to ENGINEERING DESIGN or TESTS OF MEMBERS AND COMPLETED STRUCTURES thus avoiding an oversight by the searcher of important allied data.

The readiest use may be made of this index by gaining some familiarity with the main classifications as follows:

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